



Über dieses Buch

Dies ist ein digitales Exemplar eines Buches, das seit Generationen in den Regalen der Bibliotheken aufbewahrt wurde, bevor es von Google im Rahmen eines Projekts, mit dem die Bücher dieser Welt online verfügbar gemacht werden sollen, sorgfältig gescannt wurde.

Das Buch hat das Urheberrecht überdauert und kann nun öffentlich zugänglich gemacht werden. Ein öffentlich zugängliches Buch ist ein Buch, das niemals Urheberrechten unterlag oder bei dem die Schutzfrist des Urheberrechts abgelaufen ist. Ob ein Buch öffentlich zugänglich ist, kann von Land zu Land unterschiedlich sein. Öffentlich zugängliche Bücher sind unser Tor zur Vergangenheit und stellen ein geschichtliches, kulturelles und wissenschaftliches Vermögen dar, das häufig nur schwierig zu entdecken ist.

Gebrauchsspuren, Anmerkungen und andere Randbemerkungen, die im Originalband enthalten sind, finden sich auch in dieser Datei – eine Erinnerung an die lange Reise, die das Buch vom Verleger zu einer Bibliothek und weiter zu Ihnen hinter sich gebracht hat.

Nutzungsrichtlinien

Google ist stolz, mit Bibliotheken in partnerschaftlicher Zusammenarbeit öffentlich zugängliches Material zu digitalisieren und einer breiten Masse zugänglich zu machen. Öffentlich zugängliche Bücher gehören der Öffentlichkeit, und wir sind nur ihre Hüter. Nichtsdestotrotz ist diese Arbeit kostspielig. Um diese Ressource weiterhin zur Verfügung stellen zu können, haben wir Schritte unternommen, um den Missbrauch durch kommerzielle Parteien zu verhindern. Dazu gehören technische Einschränkungen für automatisierte Abfragen.

Wir bitten Sie um Einhaltung folgender Richtlinien:

- + *Nutzung der Dateien zu nichtkommerziellen Zwecken* Wir haben Google Buchsuche für Endanwender konzipiert und möchten, dass Sie diese Dateien nur für persönliche, nichtkommerzielle Zwecke verwenden.
- + *Keine automatisierten Abfragen* Senden Sie keine automatisierten Abfragen irgendwelcher Art an das Google-System. Wenn Sie Recherchen über maschinelle Übersetzung, optische Zeichenerkennung oder andere Bereiche durchführen, in denen der Zugang zu Text in großen Mengen nützlich ist, wenden Sie sich bitte an uns. Wir fördern die Nutzung des öffentlich zugänglichen Materials für diese Zwecke und können Ihnen unter Umständen helfen.
- + *Beibehaltung von Google-Markenelementen* Das "Wasserzeichen" von Google, das Sie in jeder Datei finden, ist wichtig zur Information über dieses Projekt und hilft den Anwendern weiteres Material über Google Buchsuche zu finden. Bitte entfernen Sie das Wasserzeichen nicht.
- + *Bewegen Sie sich innerhalb der Legalität* Unabhängig von Ihrem Verwendungszweck müssen Sie sich Ihrer Verantwortung bewusst sein, sicherzustellen, dass Ihre Nutzung legal ist. Gehen Sie nicht davon aus, dass ein Buch, das nach unserem Dafürhalten für Nutzer in den USA öffentlich zugänglich ist, auch für Nutzer in anderen Ländern öffentlich zugänglich ist. Ob ein Buch noch dem Urheberrecht unterliegt, ist von Land zu Land verschieden. Wir können keine Beratung leisten, ob eine bestimmte Nutzung eines bestimmten Buches gesetzlich zulässig ist. Gehen Sie nicht davon aus, dass das Erscheinen eines Buchs in Google Buchsuche bedeutet, dass es in jeder Form und überall auf der Welt verwendet werden kann. Eine Urheberrechtsverletzung kann schwerwiegende Folgen haben.

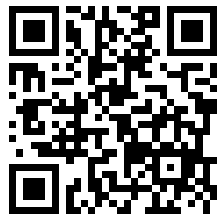
Über Google Buchsuche

Das Ziel von Google besteht darin, die weltweiten Informationen zu organisieren und allgemein nutzbar und zugänglich zu machen. Google Buchsuche hilft Lesern dabei, die Bücher dieser Welt zu entdecken, und unterstützt Autoren und Verleger dabei, neue Zielgruppen zu erreichen. Den gesamten Buchtext können Sie im Internet unter <http://books.google.com> durchsuchen.

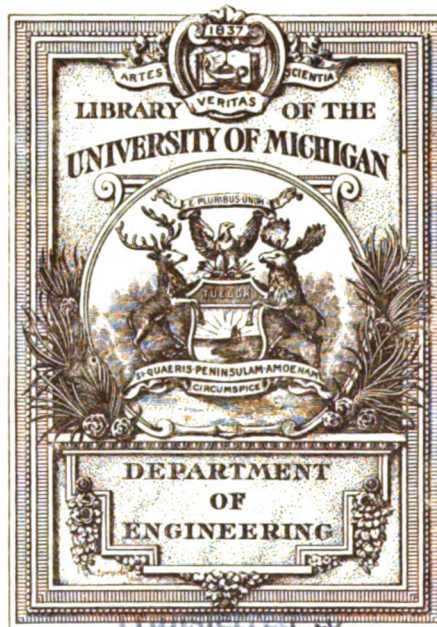
This is a reproduction of a library book that was digitized by Google as part of an ongoing effort to preserve the information in books and make it universally accessible.

GoogleTM books

<https://books.google.com>



CONCRETE
AND
CONSTRUCTIONAL ENGINEERING



GENERAL LIBRARY

U

GEN. LIBRARY.

TA
680
.C78



22



View of Finished Stand
REINFORCED CONCRETE RACK-COURSE STAND, CRITENHAM.
(For description see page 3.)

E 56

CONCRETE AND CONSTRUCTIONAL ENGINEERING

VOL. X.

1915.

Nos. 1 to 12.

**A MONTHLY JOURNAL FOR ENGINEERS,
ARCHITECTS, SURVEYORS & CONTRACTORS
and all interested in CEMENT, CONCRETE,
REINFORCED CONCRETE, FIRE-RESISTING
CONSTRUCTION and STRUCTURAL STEEL.**



VOLUME X.

All Rights Reserved

*Published at 4 Catherine Street, Aldwych, London, W.C.
Editorial Offices at : 8 Waterloo Place, Pall Mall, London, S.W.*

1915

CONCRETE AND CONSTRUCTIONAL ENGINEERING

VOLUME X. 1915.

Published at 4, Catherine Street, Aldwych, London, W.C.

Editorial Offices at: 8, Waterloo Place, Pall Mall, London, S.W.

INDEX.

	PAGE		PAGE
BOOKS, NEW:		Microscope in the Study of Concrete, The	272
Advisory Committee on Rural Cottages, Report of the ...	215	Municipal Works, Concrete and Reinforced Concrete in ...	61
British Standard Specification for Portland Cement, Revised 1915 ...	258	Obituary Notice: The late Mr. Edward Coignet ...	221
Concrete Roads and Kerbs ...	107	Oil-mixed Concrete ...	481
Concrete and Steel Construction (Part I.), Buildings. By H. T. Eddy and C. A. P. Turner ...	106	Research Work, The Need for ...	431
Concrete Stone Manufacture. By Harvey Whipple ...	259	Rural Housing ...	115
Construction of Masonry Dams. By Chester W. Smith ...	424	Shipbuilding Work, Portland Cement Concrete and Cement Concrete in ...	482
Design of Steel Bridges. By F. C. Kunz, C.E. ...	476	Wind Pressure, The Question of ...	219
Engineering Society of China: The Report of the Special Committee on Reinforced Concrete ...	54	GENERAL:	
National Physical Laboratory: Report for the Year 1914-15 ...	476	Action of Boiling Water on Concrete, The. By Prof. E. R. Matthews, A.M.Inst.C.E. ...	29
Plain and Reinforced Concrete Arches. By J. Melan ...	424	Annual General Meeting: Concrete Institute Bridges on the Columbia Highway, U.S.A., Some Concrete. By K. P. Billner ...	404
Recapture and Expansion of British Trade: The Building Trade ...	477	Bridge near Merthyr, Reinforced Concrete Bristol General Hospital, Reinforced Concrete at the. By Albert Lakeman, M.S.A. ...	169
Reinforced Concrete in Practice. By A. Alban H. Scott ...	308	Buoys and Sinks, Concrete ...	575
Spon's Architect's and Builder's Pocket Price Book ...	258	Caissons at Sunderland Docks, Reinforced... Column Bending Moments in Frame Construction. By Ewart S. Andrews, B.Sc., Eng. (Lond.) ...	10, 68
Strength of Materials, The. By E. S. Andrews ...	590	Concrete Columns and Their Cost. By Rohinton M. Fram Mirza ...	132
Structural Engineer's Handbook. By Milo S. Ketchum, C.E. ...	106	Concrete Institute, The ...	122, 283, 609
Ternary System, The ...	259	Concrete for Ornamental Garden Work ...	32
Text-book on Practical Mathematics for Advanced Technical Students. By H. Leslie Mann, B.Sc. ...	648	Concrete Road Construction ...	146
Wind Stresses in the Steel Frames of Office Buildings. By W. M. Wilson and G. A. Mancy ...	590	Concrete Roads ...	560
CEMENT:		Correspondence (see Memoranda), also:	
British Standard Specification for Portland Cement ...	597	Cottages at Crayford, Concrete ...	108, 309, 536
Durability of Cement Drain Tiles in Alkali Soils ...	599, 628	Crossley Lads' Club, Manchester, Reinforced Concrete at the ...	483
Geographical Model Constructed in Cement, A ...	626	Dam of Medina Valley Irrigation Project in Texas, Main Concrete Dam and Diversion ...	126
EDITORIAL NOTES:		Deep Water Concreting through a Tube. By E. B. Van de Gryn ...	39
British Standard Specification for Portland Cement ...	597	Determination of the Position of the Neutral Axis in a Tee-Beam by the Aid of Simple Graphs, The. By H. E. Lance Martin, B.Sc. (Lond.) ...	333
Cement Drain Tiles in Alkali Soils ...	599	Durability of Cement Drain Tiles in Alkali Soils. By R. J. Wig and G. M. Williams ...	628
Concrete Institute, The. The Annual Meeting ...	260	Earthquake Districts, Reinforced Concrete in. Regulations by the Ministry of Public Works, Rome ...	237
Construction and Protection of Buildings in Relation to Fire ...	115	Effect of Reinforced Concrete upon Architectural Design, The. By Frank N. Heaven, A.R.I.B.A. ...	433
Contractors and Reinforced Concrete ...	321	Empire and India Houses, Kingsway. By A. Lakeman, M.S.A. ...	273
Earthquake Districts, Reinforced Concrete in ...	62	Enquiries (See Memoranda).	
Economy in Reinforced Concrete Construction ...	167	Fixing Temporary Blinds to Reinforced Concrete Beams ...	522
Government Certification of Cement intended for Export ...	600	Ford Motor Co., Chicago. A Reinforced Concrete Building for the ...	77
Influence of the War, The ...	543	Gas Works at Keilhaven, Rotterdam, Reinforced Concrete in the New ...	337
London Building Acts, The ...	271	Gate Structures for Irrigation Canals, Concrete in ...	455
London County Council Regulations for Reinforced Concrete Construction ...	377	Geographical Model Constructed in Cement, A ...	626

	PAGE
Halifax Ocean Terminals. By A. F. Dyer, A.M.Inst.C.E.	194
High Strength Concretes Produced through Lowering of Surface Tension of Mixing Water. By Nathan C. Johnson	615
Influence Lines for Beam Deflections. By Ewart S. Andrews, B.Sc., Eng.	393
Island Site Hotel, The. By Albert Lake-man, M.S.A.	545
Lightning on a Steel Dome and Reinforced Concrete Walls, An Instance of ...	93
London's Reinforced Concrete Regulations (as Revised) ...	441, 502, 507
Marine Drive, Exmouth ...	503
Mechanical Disintegration of Defective Concretes. By Nathan C. Johnson ...	348
Microscope as an Aid in Proportioning Concrete for Strength. By Nathan C. Johnson ...	398
Microscope as a Check on Construction. By Nathan C. Johnson ...	448
Microscope shows Importance of Mixing as a Factor in making Strong Concrete ...	514
Microscope in the Study and Investigation of Concrete, The. By Nathan C. Johnson ...	288, 348, 398, 448, 514, 615
Moment of Inertia of Reinforced Concrete Sections. By John Davenport, M.Sc. ...	179
New Exchange Buildings, Swansea, The ...	117
Oil Mixed Portland Cement Concrete ...	492
Ornamental Garden Work, Concrete for ...	32
Padstow Station, Reinforced Concrete Road ...	505
Patents relating to Concrete, Recent British ...	23, 187, 344, 508
Pier at Padstow, Reinforced Concrete. By Sir G. Croydon Marks, M.P. A.M.Inst.C.E.	223
Port Dundas Distillery. By Albert Lake-man, M.S.A.	381
Power House, New Zealand, Reinforced Concrete. By H. C. Gayford ...	17
Presidential Address, Concrete Institute ...	609
Problems in the Theory of Construction ...	10, 68, 230, 303, 555
Quay Wall, Southampton, Reinforced Concrete ...	233
Racecourse Stand, Cheltenham, Reinforced Concrete. By Albert Lakeman, M.S.A. ...	3
Regent Palace Hotel ...	545
Roads, Concrete ...	500
Road Construction, Concrete ...	140
Road at Mountnessing, Essex ...	502
Road at Whitefriars, Chester, The New ...	501
Roberts Lane, Saltney, Chester ...	500
Semi-circular Arch carrying its own weight only. By Ewart S. Andrews, B.Sc. ...	230
Standard Specification for Reinforced Concrete Work, A ...	122
War in Relation to Building, The ...	243
Wharf and Jetty for the Port Talbot Railway and Dock Co., New Reinforced Concrete ...	601
What is a 1:2:4 Concrete? By H. C. Johnson ...	85
Wind Stresses in Building Frames ...	555
INDUSTRIAL NOTES:	
Interlocking Blocks, New System of ...	533
Steel Piling ...	260
New Method of Reinforced Concrete Sheet Piling, A ...	310
MEMORANDA:	
Acid Towers Built of Concrete ...	267
Aeroplane, Reinforced ...	538
American Concrete Institute ...	57
Association of Consulting Engineers (Incorporated), The ...	57
Belfast Harbour Equipment ...	112
Blocks at Fishguard, Concrete ...	427
Fish Harbour, Improvements at ...	521
Bridge, Devon ...	375
Bridge, New, Essex ...	506
Bridges, Thames, New Reinforced Concrete ...	112
British Docks ...	112
British Engineers' Association, The ...	57
Canadian Engineering Operation, A ...	506
Cathedral in Reinforced Concrete at Georgetown, Demerara ...	59

	PAGE
Cement as Anti-Sand Blast ...	539
City and Guilds of London Institute, Department of Technology ...	479
Colliery Electric Lamp Rooms ...	216
Concrete Fence Flank Transit Route, Four and a half miles of ...	653
Concrete Institute, The ...	049
Concrete Scow has withstood 45 years' Hard Service ...	591
Concrete Surfaces ...	267
Contracts Accepted ...	320, 544
Contracts Open ...	268, 320
Drain Head, Concrete ...	113
Draining Land by Wells, Concrete Drain Head ...	113
Dundee New City Hall ...	532
Effectual Sealing of Water Dams by Cementation, The ...	316
Engine Foundation, How to Build an ...	655
Enquiries ...	59
Erecting Gas Engines ...	593
Errata Notes ...	376, 430, 480
Expenditure on Public Works ...	317
Fires due to Air Raids ...	315
Fishguard Harbour Works ...	553
Flagpole at Panama, Reinforced Concrete ...	592
Footbridge, Neasden, Reinforced Concrete ...	479
Forcing Frames in Reinforced Concrete ...	653
Grape Vine Posts of Concrete ...	655
Hostile Air Raids, Police Warning ...	049
Houses for the Working Classes ...	050
Institute of Arbitrators ...	264
Institution of Civil Engineers, The ...	164
Iron and Steel Institute ...	164, 264
Keel of a Sailing Craft, Concrete for the ...	539
Kehoe & Co. Simplex Concrete Piles, Ltd. ...	216
King George Hospital, The ...	165
Law and Engineering ...	582
Lighthouse, Reinforced Concrete ...	541
Lining the Shafts of Mines, a Downward System ...	650
Lining large Steel Coal Tank with Concrete ...	58
Lining for Steel Hull of a Steam Launch, Concrete ...	59
Literary Note ...	319
Loans for Public Work ...	375
L.C.C. Regulations with respect to the Construction of Buildings wholly or partly of Reinforced Concrete, The ...	426
Loughor Bridge ...	165
Manchester Municipal School of Technology, The ...	538
Microscope in the Study of Concrete, The ...	559
Midland Institute, Birmingham ...	049
Motor Track in America, First Concrete ...	506
National Road Conference and Exhibition, 1915, The ...	315
National Road Exhibition ...	429
Nairobi Stock Exchange Buildings ...	317
New Pennsylvania Elevator of Reinforced Concrete at Philadelphia ...	264
Oil-mixed Concrete Tests ...	651
Paving Streets with Concrete ...	501
Picture Houses, Reinforced Concrete ...	375
Pictures Showing Concrete Construction, Moving ...	505
Pier at Tiree, Long Concrete ...	502
Pipes, Reinforced Concrete ...	541
Plugs, Concrete ...	480
Police Warning, Hostile Air Raids ...	049
Polish on Concrete Surfaces ...	541
Porch at Elkhart, Indiana, A Concrete ...	505
Port of London Authority, New Cold Stores ...	592
Proposed New Works ...	267, 319, 375, 429, 541, 657
Publications Received ...	114, 166, 218, 541
Purifier House at the Poole Gas Works, Concrete for ...	59
Reinforced Concrete Regulations ...	559
River Trent, the Improvement of the ...	165
Roads at Chiseldon, Wilts, Reinforced Concrete ...	539
Road Making, Cheaper ...	539
Safes, Reinforced Concrete for Bank ...	654
San Francisco Shore Protection ...	654
Shanghai Electricity Works ...	654
Sleepers for Tramways, Reinforced Concrete ...	112, 373
Slipway for Motor Boats, the Construction of a ...	112
Small Holdings, Concrete Buildings for ...	316
Smallpox Hospital, Whitehaven ...	512
Surface Imperfections ...	480

	PAGE		PAGE
Tenders Accepted	268, 376, 429	Shear and Problems arising therefrom.	
Tests of Six Types of Reinforced Concrete		By H. Kempton Dyson	43
Fence Posts	316	Some Modern Methods of Arch Calcula-	
Trade Notices	218, 430, 542, 596	tion. By Ewart S. Andrews, B.Sc. ...	151
Tramway Sleepers, Concrete	112, 373	Stability of Quay Walls on Earth	
University of London	538	Foundations. By F. E. Wentworth-	
Vats for Olive Oil, Concrete	653	Sheilds, M.Inst.C.E.	358
Vaults, Concrete	59	Tests of Reinforced Concrete Structures	
Warrington Bridge	264	on the Great Central Railway. By J.	
Westminster Technical Institute	538	Benjamin Ball, M.Inst.C.E.	46
White City, The	264	Wind Pressure, Some Notes on. By R.	
Zeppelin Raids, Police Warning	538	Graham Keevil, A.M.I.Mech.E.	246
NEW WORKS:			
Bridge at Allentown	53	REINFORCED CONCRETE:	
Bridge over the River Calder at Sterne		Bridge over River Calder. Reinforced	
Mills, near Sowerby Bridge, Halifax,		Concrete	410
Yorks, Reinforced Concrete	419	Bridge at Clacton-on-Sea, Ornamental Rein-	
Bridge, Clacton-on-Sea, Ornamental Rein-		forced Concrete	304
forced Concrete	304	Bridges on the Columbia Highway, U.S.A.,	
Bridges, Coventry Loop Line, London and		Some Concrete	494
North-Western Railway, Reinforced Con-		Bridges, Coventry Loop Line, L. and	
crete	368	N.W. Ry., Reinforced Concrete	368
Buildings in China, Reinforced Concrete ...	103	Bridge near Merthyr	323
Church Hastings, New Zealand, Reinforced		Bristol General Hospital, Reinforced Con-	
Concrete	372	crete at the	169
Cleckheaton Gas Works	395	Buildings in China, Reinforced Concrete...	103
Column Bridge at Hudson, Wisconsin,		Caissons at Sunderland Docks, Reinforced	
The Concrete	212	Concrete	63
Concrete Cottages at Norwich, Some ...	644	Church, Hastings, N.Z., Reinforced Con-	
Cottages at Wrotham, Concrete	254	crete	372
Electrification of the London and South-		Cleckheaton Gas Works	395
Western Railway (Suburban Lines), Rein-		Contractors and Reinforced Concrete ...	321
forced Concrete in the	583	Crossley Lads' Club, Manchester, Rein-	
Gallery at the Empire Theatre, Whitby,		forced Concrete at the	483
Description of Reinforced Concrete ...	253	Earthquake Districts, Reinforced Concrete	
Indo-China Bank, Singapore, New Offices		in	237, 62
for the	420	Economy in Reinforced Concrete Construc-	
Large Concrete Arch Culvert, A	475	tion	207, 167
Massachusetts Institute of Technology,		Effect of Reinforced Concrete upon Archi-	
The New Building	159	tectural Design	433
New Offices for the Indo-China Bank,		Electrification of the London and South-	
Singapore	420	Western Railway (Suburban Lines),	
New York State Barge Canal, Concrete and		Reinforced Concrete in the	583
Electric Work on	528	Engineering Society of China, The Report	
Parksville Concrete Dam and Power Plant		of the Special Committee on Reinforced	
Sub-structure of the Lethbridge Viaduct,		Concrete	54
Canada	641	Ford Motor Co., Chicago, A Reinforced	
Wigan Baths, Millgate, Extension and		Concrete Building for the	77
Alterations of the	473	Fixing Temporary Blinds to Reinforced	
		Concrete Beams	522
		Gallery at the Empire Theatre, Whitby.	
		Description of Reinforced Concrete ...	253
		Lightning on a Steel Dome and Reinforced	
		Concrete Walls, An Instance of	83
		London County Council Regulations for	
		Reinforced Concrete Construction	
			377, 441, 502, 507
		Massachusetts Institute of Technology,	
		The New Building	159
		Moment of Inertia of Reinforced Con-crete	
		Sections	179
		Municipal Works, Concrete and Reinforced	
		Concrete in	61
		New Exchange Buildings, Swansea, The ...	117
		New Gas Works at Keilhaven, Rotterdam,	
		Reinforced Concrete in the	337
		Padstow Station, Reinforced Concrete ...	505
		Parabolic Reinforced Concrete Arch ...	525
		Pier at Padstow, Reinforced Concrete ...	221
		Plain and Reinforced Concrete Arches	
		Points respecting Reinforced Concrete for	
		Roads, Sewerage, etc.	523
		Port Dundas Distillery	381
		Power House, New Zealand, Reinforced	
		Concrete	17
		Quay Wall, Southampton, Reinforced Con-	
		crete	233
		Racecourse Stand, Cheltenham, Reinforced	
		Concrete	3
		Reinforced Concrete Construction	49
		Reinforced Concrete in Practice	308
		Sheet Piling, A New Method of Reinforced	
		Concrete	310
		Standard Specification for Reinforced Con-	
		crete Work, A	122
		Tests of Reinforced Concrete Structures on	
		the Great Central Railway	46
		Wharf and Jetty for the Port Talbot Rail-	
		way and Dock Co., New Reinforced	
		Concrete	601
		Wigan Baths, Millgate, Extensions and	
		Alterations of the	473

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X., No. 1.

LONDON, JANUARY, 1915.

EDITORIAL NOTES.

1915.

It is useless to ignore the fact that concrete and reinforced concrete have, during the past year, been somewhat "marking time," and for all concerned it would be well to understand this plainly with a view to finding some remedy.

The primary cause goes back some two or three years, when a tendency to exaggerate the utility of these materials, both from the practical point of view and that of economy, brought about a certain reaction, and by the very strenuous efforts of the steel frame propagandists, who were only too pleased to foster any feeling of disappointment.

The professional man and also the leaders of the industries concerned only too keenly resent exaggerated claims, as the true claims are sufficiently wide and proved by experience.

Unfortunately the Concrete Institute, which was intended to do so much for concrete and reinforced concrete, and started doing it so well, has not been able to give to the true advancement of the subject that undivided and strenuous attention which had been anticipated.

What with the bickerings within the Institute on matters of management and administration, what with the energetic set made by the steel frame interests to obtain control of the Institute and the desire of certain of the members for special prerogatives, the principal objects of the Institute were at times almost entirely lost sight of, and had it not been for the outbreak of war, whereby all contentious questions were shelved, the year 1914 might easily have seen the disbandment of the Concrete Institute or its departure into other realms to make way for some new and more virile institution.

What we require at the present moment to wipe away the stagnation which has made itself felt in matters relating to concrete and reinforced concrete is, on the professional side, an active indication, provable by facts, of the good work done and the economies effected and advantages obtained by concrete and reinforced concrete in the past decade. The fine work accomplished requires description; examples of success, honest analysis; interesting facts, full as distinct from secretive explanation; rather than any more arguments as to various mathematical conundrums, algebraical notation, and possible regulations in connection with this subject. The whole of the subject has so many practical elements and interests that it will appeal to all concerned more readily if examples are more actively put forward, discussed, analysed, and urged as useful precedents.

On the industrial side the same holds good, and we want more actual facts as to what has been done and what economies have been so splendidly

effected rather than dicta of the "might-have-beens" or future possibilities. Those concerned on the industrial side should plainly face the fact that there are certain limitations to concrete and reinforced concrete, just as there are for any other form of construction.

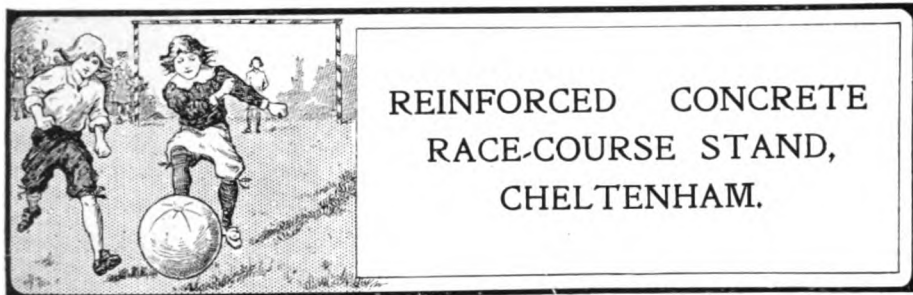
But if we have spoken to the professional man and the industrial leader, there are other classes to whom we would also like to say a word.

The first are the makers of all those forms of machinery so useful in concrete work. Be it the maker of the block-making machine or the concrete mixer or what not, it lies in his hands, too, to be more precise in indicating by examples what his machines have achieved, rather than to make statements as to the achievements and output he anticipates. He should simplify his literature, and even limit the number of types of appliances he puts on the market so as not to confuse users, and, above all, he should remember that in times like the present he can not only assist materially in the use of concrete and reinforced concrete at home, but particularly in India and in the Colonies, in the South American Republics, and elsewhere, thereby indirectly not only benefiting himself, but also the professions and trades of the home country.

Lastly, we would speak to the purveyors of the materials of which concrete is composed. There is little we can say about the great cement industry of this country as far as improvement in policy is concerned. It fortunately happens to be one of those industries that is conducted on thoroughly national, we may even say international lines, and the splendid advancement of the product of Portland cement has been the greatest auxiliary in the development of concrete and reinforced concrete during the past decade. This industry, too, can be trusted to hold its own at home, in the Colonies, and abroad. Its work has been a credit to our manufacturing activities.

But there are various purveyors: those who provide stone, sand, clinker, coke breeze, etc., materials which all play a part in the production of concrete and reinforced concrete. These have not as yet quite lived up to modern requirements as a whole, although there are one or two notable exceptions, and the same must be said of those local purveyors in many of our Colonies. The makers of bars and mesh work can likewise, with perhaps one or two notable exceptions, work on broader lines. It is almost scandalous that professional men and leaders in the industry should be constantly handicapped by unsatisfactory deliveries of the materials known as aggregates, and so often have to encounter delays from the bar and mesh makers.

Generally speaking, all concerned in concrete and reinforced concrete—the architect, the engineer, the quantity surveyor, the specialist designer, the specialist contractor, the ordinary contractor, and the purveyor—should realise that they are concerned in one of the greatest constructional developments that have been seen during the last 50 years; that between them they have the making of one of the greatest industries within sight at the present moment, and that the time has arrived for more active co-operation between all the interests concerned and for a fixed policy as to development at home, in the Colonies, and certain foreign markets.



By ALBERT LAKEMAN, M.S.A.

We have already shown in previous issues that reinforced concrete is well suited and has many advantages to structures such as the one here described, and the present example shows that the material also lends itself well to architectural treatment when used in this connection.—ED.

THIS race-course stand is an excellent example of an architectural reinforced concrete structure, and indicates what can be accomplished when the work is designed by architects who are in sympathy with the material and who take advantage of the possibilities which it lends to simple, pleasing lines expressive of the constructional value of the material employed. The finished stand is illustrated in our frontispiece, and it is undoubtedly one of the most pleasing

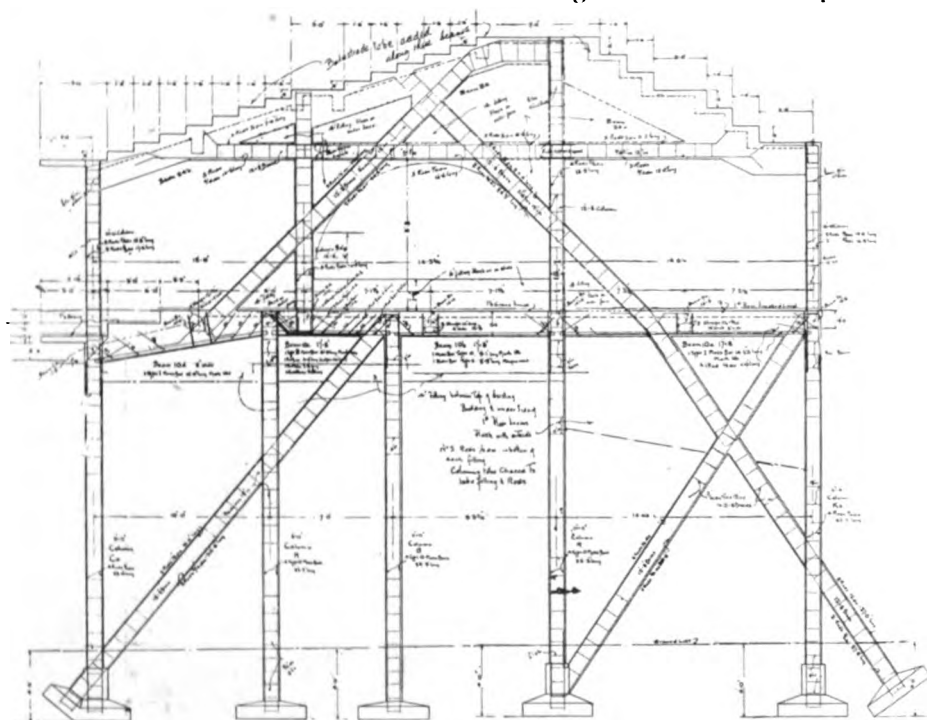


Fig. 1. View of Balcony on First Floor,
REINFORCED CONCRETE RACE-COURSE STAND, CHELTENHAM.

stands in this country, and was completed about twelve months ago. The architects were Messrs. Chatters and Smithson, F.R.I.B.A., of Cheltenham, and the contractors for the whole of the structural work were Messrs. William Moss and Sons, Ltd., of Loughborough, while the details of the reinforced concrete work were prepared by Mr. H. M. de Coleville, A.R.I.B.A., of 48, Bedford Row, London, W.C., to suit the requirements of the architects. The stand is rather unique in the fact that the

topmost tier is arranged in such a manner that spectators can obtain a view of the racing from both back and front of the stand, this being brought about by the position of the building in relation to the race-course, the latter curving away to the rear of the structure with a wide sweep.

The stand has a total length of 90 ft. and a width of 44 ft., and the ground floor is arranged as a large court with an open front and enclosed on three sides only. Two large open fireplaces are placed back to back in the centre of the court, and a wide staircase leads up to the upper part of the stand, while access is obtained from the club enclosure to the paddock through an entrance at the back of the court. On the first floor a large luncheon room is provided,



Section on C.D.

Fig. 2. Cross-Section showing Braces
REINFORCED CONCRETE RACE-COURSE STAND, CHELTENHAM.

this being 40 ft. long and 23 ft. wide, finished with a barrel ceiling executed in fibrous plaster with suitable enrichments. An angle-nook is formed at one end of the room, and the walls and columns are treated with panelled and fluted wainscoting. A view of the race-course and paddock can be obtained from the balconies, which are arranged on either side of this room, which is provided for the use of the committee and stewards. The remainder of the first floor is allocated to private luncheon rooms, with the necessary offices for service, boxes, and balconies.

The constructional work was carefully planned and detailed to suit the architectural treatment of the stand, and the columns were spaced at about



Fig. 3. Construction of Roof Stand.
REINFORCED CONCRETE RACE-COURSE STAND, CHELTENHAM.

14-ft. centres, both longitudinally and transversely, thus giving six main longitudinal rows and four transverse rows. One end of the stand is, however, somewhat irregular in outline owing to some projecting portions, and the spacing of the columns was modified as necessary to suit these projections. In addition to the vertical supporting columns several diagonal struts or braces were introduced in order to stiffen the structure and render it perfectly rigid and free from vibration, and a section of the stand showing the constructional members is illustrated in *Fig. 2*, where several of the braces can be seen. It might appear at first sight that these would be detrimental to the stand from a spectator's point of view, but they were all carefully considered as regards the positions, and in every case they are arranged to come within the partitions or in such a position that they do not interfere either with the view or the decorative treatment. The effective bracing of a structure of this kind is undoubtedly of great importance, owing to the exposed position and the nature of the load, and this has been fully considered by the designer.

The foundations to the columns and braces are carried down to a level 4 ft. below the ground line, and in some cases the two are combined below the ground level to come upon one foundation slab, and in other instances they are separate. In the drawing reproduced in *Fig. 4* an example is given of the combination of columns and braces, and also a plain column foundation. The latter are generally 4 ft. square, the largest employed being

4 ft. 6 in. square, and these have a minimum thickness of 9 in. at the extreme outer edges, increased by sloping up the concrete towards the column to give a maximum of 12, 15, or 18 in. according to the load. In addition to the increase in thickness obtained by sloping up the concrete, the intersection between the column shaft and the base is covered with a block of concrete usually 16 in. square and about 18 in. high, an example of which is shown in the detail above referred to. The reinforcement in these slabs was composed of a lattice of $\frac{5}{8}$ -in. and $\frac{3}{4}$ -in. rods in the lower surface, having a cover of 2 in.

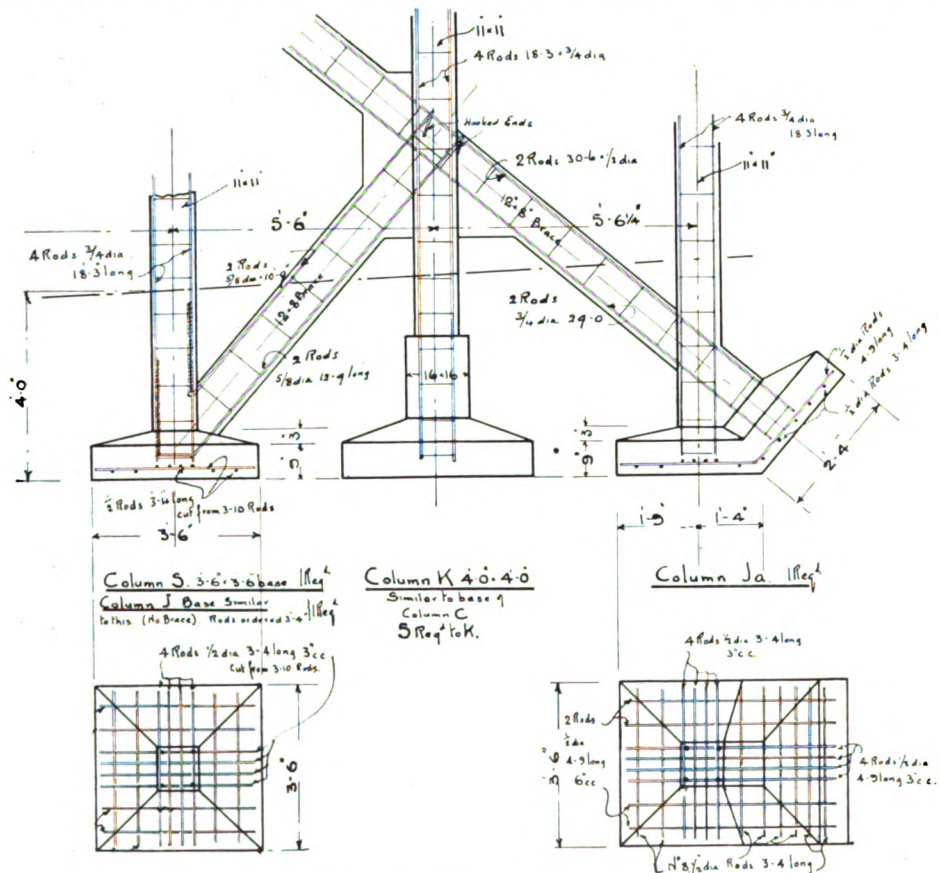


Fig. 4. Showing Foundations to Columns and Braces.
 REINFORCED CONCRETE RACE-COURSE STAND, CHELTENHAM.

of concrete, these rods being spaced at 4-in. or 6-in. centres. When the foundation to the brace occurred close to the column slab it was combined with it, as shown in the drawings, by forming a canted slab, part of which was horizontal under the column and the other part placed at right angles to the length of the brace to transmit the thrust directly to the soil.

All the columns up to the first floor are 11 in. square, and above this they are reduced to 10 in. square. They are all reinforced with four lines of vertical reinforcement, consisting of $\frac{5}{8}$ -in. diameter rods above the first floor, and

either $\frac{3}{4}$ -in. diameter rods or Moss bars below this level, the latter being placed with their webs on the diagonal lines joining the opposite corners. The whole of the vertical rods are tied with $\frac{3}{8}$ -in. diameter links at 9-in. and 12-in. centres.

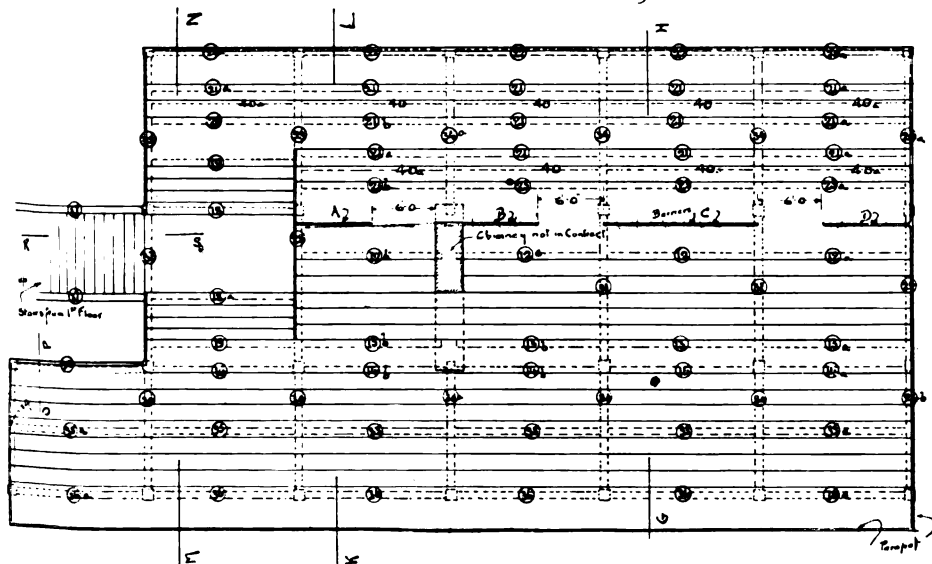


Fig. 5. Second Floor Plan.

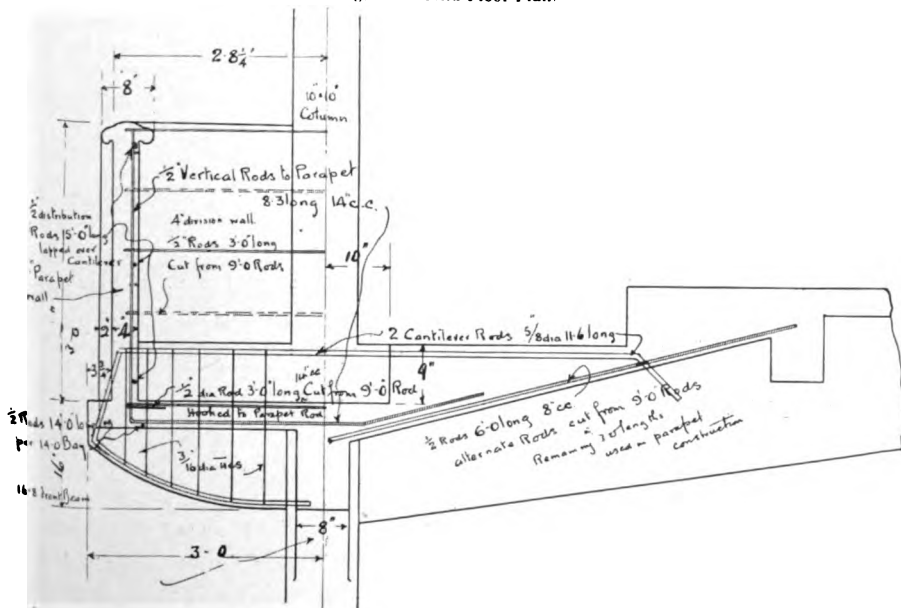


Fig. 6. Section of First Floor Balcony.
REINFORCED CONCRETE RACE-COURSE STAND, CHELTENHAM.

The braces are all 12 in. by 8 in., reinforced with four main rods tied with links at 12-in. centres. The total height of the stand from the ground level is about

35 ft., and the braces are arranged in two heights in some positions. The main beams at the first floor level have a maximum span of 14 ft., and these vary in size considerably, the largest being 18 in. deep and 10 in. wide, reinforced with two Moss bars and having stirrups throughout the length.

The floor is divided up into bays, about 14 ft. square, by these main beams, and generally one secondary beam is introduced into each bay to cut up the span for the floor slab into 7 ft., and these secondary beams are generally 14 in. deep and 8 in. wide, reinforced with one bar in the lower surface; and stirrups as for the main beams.

The slabs are 4 in. thick, and they are reinforced with $\frac{1}{2}$ -in. rods at 8-in. pitch, and $\frac{1}{2}$ -in. diameter continuity rods are placed in the upper surface where passing over the beams.

The front balcony is reached from the first floor level by two steps of 9 in. each, and as the level of the balcony is thus 18 in. below the main floor level, the main beams carrying the adjoining bay had to be somewhat varied to suit the levels of the steps. The beams are 8 in. wide, and they are constructed with a raking soffit and a varying depth, and they are reinforced with three bars in the lower surface. The balcony itself has a projection of 2 ft. 8 in. from the centre line of the outer row of columns to the outside face of the balustrade, and the floor is formed with concrete, 4 in. thick, reinforced with $\frac{1}{2}$ -in. diameter rods.

The floor is carried by cantilevers placed at each column, these being 8 in. wide and having a maximum depth of 25 in., 12 in. of which occurs under the balcony floor and is visible in the finished work as a bracket. These cantilevers are reinforced with two $\frac{3}{4}$ -in. diameter rods, which are carried along the top of the member and continued down and around to the underside and back to the supporting column. Ties, $\frac{3}{8}$ -in. diameter, are passed right round the rods between the two surfaces, and the rods at the top are cranked down into the adjoining main beam.

The parapet to the balcony is of reinforced concrete, 3 ft. high and 4 in. thick, with small projecting piers over each cantilever, 10 in. wide and 2 in. projection. The parapet is reinforced with $\frac{1}{2}$ -in. vertical rods at 14-in. centres, these being bent at right angles at the bottom and carried back in the floor behind the columns, and three horizontal distribution rods are also provided, these being lapped over the cantilevers. The parapet is finished at the top with a moulded concrete capping, 8 in. wide and 3 in. deep. A detail of this work is illustrated in *Fig. 6*.

A back balcony, 5 ft. 6 in. wide, was also provided, but no cantilevers were required for this, as it is directly carried by 12-in. by 8-in. beams supported on the columns, the outer row of the latter finishing at this level and supporting the beams under the outer edge of the balcony. The floor of the balcony is 4 in. thick and reinforced with $\frac{1}{2}$ -in. diameter rods at 10-in. centres. A front balcony is also provided at the second floor level, which is about 24 ft. 6 in. from the ground, and this is constructed in a similar manner to that occurring at the first floor.

The second floor comprises the roof stand, where provision is made for a view from front and back, as previously mentioned.

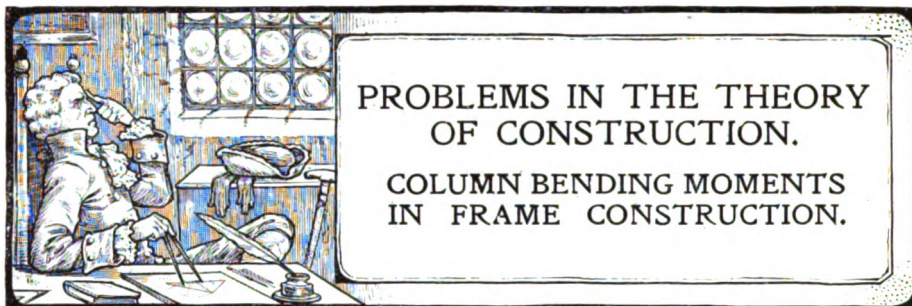
The accommodation for spectators in the front consists of twelve tiers, each 18 in. wide and with a rise of 6 in., and a level walking way, 3 ft. wide,

is formed half-way up the height and immediately over the second row of columns. At the top of these tiers a level platform, 7 ft. wide, is constructed before descending at the back, where there are four tiers 18 in. wide and 12 ft. high, and two platforms, each 3 ft. wide. The front tiers are constructed with a minimum thickness in the soffit of 4 in., and this is reinforced with $\frac{1}{2}$ -in. and $\frac{3}{4}$ -in. rods parallel to the pitch, and with continuity rods when passing over the secondary beams. The latter are formed under the edges of the intermediate walking-ways or platforms, and also in the centre of the length of the tiers, and they are carried by the main beams or the columns direct. The back tiers are constructed on a different principle owing to the greater depth of the risers, these being each designed as a beam, with a thickness of 5 in. and being reinforced with one bar in the lower surface, while the treads are $3\frac{1}{2}$ in. thick with $\frac{3}{4}$ -in. rods at 11-in. centres on the underside. The main beams, which occur at each line of columns, are varied considerably according to their positions; in some cases they are sloping and parallel to the pitch of the tiers, and in other cases they have a horizontal soffit and a varying depth caused by the rise and fall of the roof. Reinforced concrete parapets are constructed all round the roof, that on the front being supported by the balcony cantilevers and that at the back occurring over the outer line of beams, with the columns carried up to the top of the parapet to form projecting piers on the face of the wall and divide it up into panels. All these parapets are similar in construction to that illustrated in the detail of the first floor front balcony, and the finished effect can be seen in the exterior view of the stand.

This structure forms another excellent example of the applicability of reinforced concrete construction, and its value for this class of building is becoming more popular day by day. The questions of maintenance and safety against fire are of primary importance, and this material is ideal from these two stand-points, while the initial outlay required will always compare favourably with any other material which possesses the same strength and lasting properties.



Fig. 7. Dining Room.
REINFORCED CONCRETE RACE-COURSE STAND, CHELTENHAM.



By EWART S. ANDREWS, B.Sc. Eng. (Lond.), M.C.I.

The following article will probably be of interest to all designing reinforced concrete work.—ED.

ONE of the most difficult problems in the theory of construction is that of the bending moments transmitted to columns in frame construction in which, as in steel frame construction, the beams are firmly connected to the columns to secure rigidity, or, in reinforced concrete construction, the whole structure is monolithic.

Most designers have now realised that in reinforced concrete design you must allow for the reverse bending moments due to the continuity or fixity of the beams at supports, but comparatively few have realised the very important fact that bending moments are transmitted to the columns, and that such bending moments must be allowed for in scientific design. The great need in this matter at the present moment is the discovery of a method which will be sufficiently near to the results of sound theory, and yet will be sufficiently simple for practical designers to be able to adopt it. Messrs. Faber & Bowie have given an excellent treatment of the subject in their "Reinforced Concrete Design" (Arnold). Our experience, however, is that, thorough though that treatment be, it involves a mathematical treatment which many designers cannot follow. The aim of the present article is to explain from the standpoint of graphics the principal points involved in the problem, to assist in the following of the more advanced treatment, so that all designers may, if so disposed, study the question of these bending moments, which all the leading authorities of structural theory regard as of fundamental importance. It is a common source of complaint among practical designers that the exponents of theory indulge in advanced mathematics instead of common sense and the results of tests. The reply is that common sense is a very variable quantity; the history of scientific progress shows that common sense is usually common fallacy; Galileo was tortured for saying that the earth was round, whereas common sense said that it was flat. As to tests, nobody values them more than the mathematical section of engineers, and nobody knows more clearly than they the great expense and extreme difficulty in getting satisfactory results from tests. Satisfactory tests require the very best brains and plenty of apparatus, both of which are costly.

Relation between Slope of Columns and Bending Moments.

The bending moments are induced in the columns by the fact that the beams slope slightly under the deflection caused by the loads, and must transmit such slope and deflection to the columns. The columns thus act in part as vertical beams, and the relation between the slope, deflection and bending moment is given by "Mohr's Theorem," which we will state as follows :—

By treating the bending moment diagram of a beam of constant section as our imaginary load, and drawing the shear and B.M. diagrams, the shear diagram gives the slope of the beam and the B.M. diagram the deflection, each multiplied by EI (E =Young's Modulus; I =Moment of Inertia of Beam Section).

The slope of a beam is the tangent of the angle of inclination with its original direction, and for the very small angles with which we are concerned the tangent is equal to the angle measured in radians.

We will now take a column DE , which is forced to have a slope θ at one end, and consider the bending moments resulting from four possible methods of support of the other end.

Case 1. Other end fixed vertically.—As shown in Fig. 1, the end D is given a slope θ and the lower end is fixed vertically; this would happen if the lower floor was equally loaded on each side of the column, but the upper floor was loaded on the left-hand side only. In all the figures the slope is shown exaggerated for clearness. There is a B.M. $=B_D$ at D and a B.M. $=B_E$ at E ; these two B.M.'s are opposite in sign because the curvature of the beam is in the opposite direction at the two ends. Since there is no transverse load between D and E , the B.M. diagram will be a straight line between the two points; we thus get the B.M. diagram $d b a c$. We now want to find the slope θ at D in terms of the B.M.'s.

FIG. 1.

The slope at the ends will be given by the imaginary reactions there, since the imaginary shears at the ends are equal to the reactions. To simplify our calculations draw ac parallel to ed ; then we may regard the B.M. diagram as made up of the rectangle $accd$, minus the triangle abc . The imaginary load, therefore, consists of the triangle, whose area P_1 acts at its centroid in one direction, and the rectangle, whose area P_2 acts at its centre in the other direction. Let r_c =imaginary reaction at c . Then taking moments about d in the usual way we have:—

$$\begin{aligned} r_c \cdot h &= P_2 \cdot \frac{h}{2} - P_1 \cdot \frac{h}{3} \\ &= B_E h \cdot \frac{h}{2} - \frac{1}{2} (B_D + B_E) h \cdot \frac{h}{3} \\ &= \frac{h^2}{6} \{ 3 B_E - (B_D + B_E) \} \\ &= \frac{h^2}{6} (2 B_E - B_D) \\ \therefore r_c &= \frac{h}{6} (2 B_E - B_D) \end{aligned} \quad (1)$$

But by Mohr's theorem $r_c = EI_c \theta$, if I_c is the moment of inertia of the column at E and $\theta = 0$

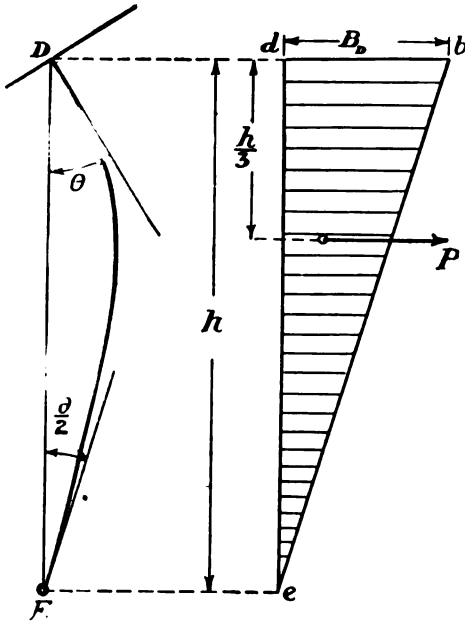


FIG. 2.

$$\therefore 0 = \frac{h}{6} (2B_E - B_D)$$

$$\text{i.e., } B_E = 2B_D \quad (2)$$

Therefore d the point of contraflexure occurs at $\frac{h}{3}$ from the bottom.

Now find r_d by taking moments about c .

$$\begin{aligned} \text{Then } r_d \cdot h &= P_2 \cdot \frac{h}{2} - \frac{P_1 \cdot 2h}{3} \\ &= B_E \cdot h \cdot \frac{h}{2} - \frac{1}{2} (B_D + B_E) h \cdot \frac{2h}{3} \\ &= \frac{h^2}{6} (3B_E - 2(B_D + B_E)) \\ &= \frac{h^2}{6} (B_E - 2B_D) \\ &= \frac{h^2}{6} (2B_D - 2B_D) \quad \text{from (2)} \\ &= \frac{h^2}{6} \left(-\frac{3B_D}{2} \right) \\ &= -\frac{h^2}{4} B_D \\ \therefore r_d &= \frac{h \cdot B_D}{4} \quad (3) \end{aligned}$$

but $r_d = EI_c \theta$

$$\text{i.e., } B_D = \frac{4EI_c \theta}{h} \quad (4a)$$

$$B_E = \frac{2EI_c \theta}{h} \quad (4b)$$

If, therefore, the angle θ is known the value of B_D can be calculated.

Case 2. Other end hinged.—The condition that the end E is hinged imposes the restriction that the B.M. is zero there. We thus get the B.M. diagram in the form of the triangle dcb shown in Fig. 2, the area P acting through the centroid at a distance $\frac{h}{3}$ from the top. By taking moments about d we have

$$\begin{aligned} r_d \times h &= P \times \frac{2h}{3} \\ &= \frac{B_D \cdot h}{2} \cdot \frac{2h}{3} \\ &= B_D \cdot \frac{h^2}{3} \\ \therefore r_d &= \frac{B_D \cdot h}{3} \\ \text{i.e., } B_D &= \frac{3EI_c \theta}{h} \quad (5) \end{aligned}$$

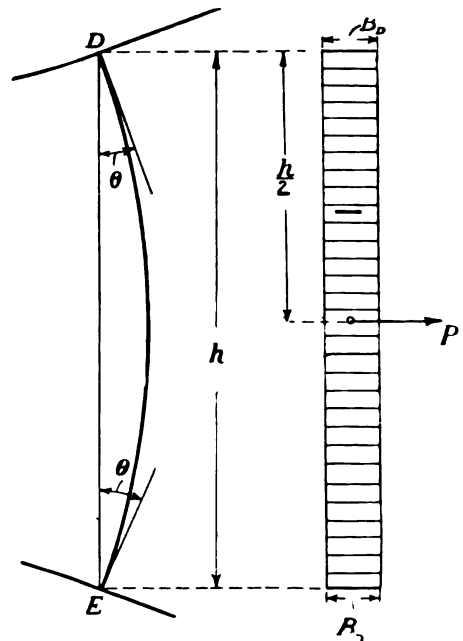


FIG. 3.

Now $r_d + r_e = P$

$$\begin{aligned}\therefore r_e &= \frac{B_D h}{2} - \frac{B_D h}{3} \\ &= \frac{B_D h}{6} \\ &= \frac{1}{2} r_d\end{aligned}$$

$$\therefore \text{Slope at } E = \frac{1}{2} \text{ slope at } D = \frac{\theta}{2}.$$

Case 3. Other end slopes equally in OPPOSITE direction.—This occurs if the upper floor is loaded on the left of the column and the lower floor equally loaded in the right. In this case the B.M.'s at the two ends must be equal, since the slopes are equal and the curvature is in the same direction. The B.M. diagram will be a rectangle, as shown, and the column will bend to a circular arc.

$$\therefore \text{each imaginary reaction} = \frac{P}{2} = \frac{B_D h}{2}$$

$$\therefore E \cdot I_c \cdot \theta = B_D \cdot \frac{h}{2} \quad (6)$$

$$\text{i.e., } B_D = B_E = \frac{2E \cdot I_c \cdot \theta}{h} \quad (7)$$

Case 4. Other end slopes equally in SAME direction.—This case is shown in Fig. 4. It would occur if both floors were loaded on left-hand side only.

The slopes at the two ends are the same, so that the B.M.'s will be the same; the curvatures are in opposite directions so that the B.M.'s will be opposite in sign.

Taking moments about e we have

$$r_d \cdot h = P \cdot \frac{5h}{6} - \frac{P \cdot h}{6} = \frac{2P \cdot h}{3}$$

$$\begin{aligned}\therefore r_d &= \frac{2P}{3} = \frac{1}{2} \cdot \frac{B_D h}{2} \\ &= \frac{B_D h}{6}\end{aligned}$$

$$\therefore EI\theta = \frac{B_D \cdot h}{6} \quad (8)$$

$$\text{i.e., } B_D = \frac{6E \cdot I_c \cdot \theta}{h} \quad (9)$$

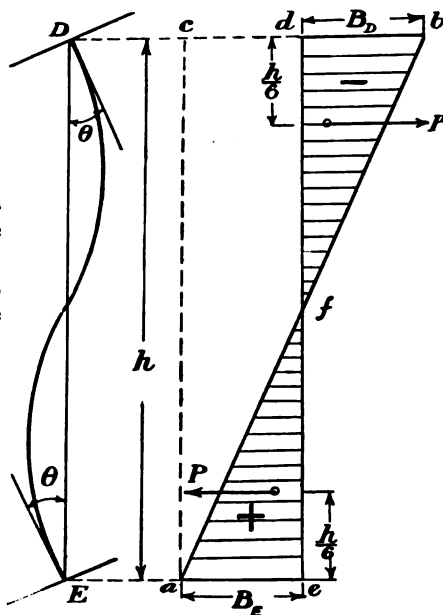


FIG 4

Summary of Results.

Case 1 $B_D = \frac{4EI_c \theta}{h}$

Case 2 $B_D = \frac{3EI_c \theta}{h}$

Case 3 $B_D = \frac{2EI_c \theta}{h}$

Case 4 $B_D = \frac{6EI_c \theta}{h}$

We learn from these four results the very important fact that the less the deflection of column for a given end slope the greater will be the B.M. and therefore the stress, because it is clear that the deflections will be the greatest in Fig. 3, next in Fig. 2, next in Fig. 1, and least in Fig. 4.

Calculation of Slopes.

In a rigid analysis of these problems we should take account of the fact that the rigidity of the columns themselves will affect the slopes which the beams will have, but an allowance for this causes great mathematical complications, except in the simplest cases. We will follow Messrs. Faber and Bowie in neglecting this as a first approximation in some cases; the result will err on the right side, but, as we shall show later, the error will often be considerable unless the beams are very rigid compared with the columns. We will first consider the case of a single span uniformly loaded and carried on columns at the ends. The B.M. diagram for the beam is a parabola as shown in Fig. 5, w being the uniform load per unit length.

$$\begin{aligned}\text{Then } r &= \frac{1}{2} \text{ area parabola} \\ &= \frac{1}{2} \cdot \frac{2}{3} \cdot l \cdot \frac{wl^2}{8} \\ &= \frac{wl^3}{24}\end{aligned}$$

$\therefore$ If E is the same for beam and column and I_B is the moment of inertia of the beam,

$$\begin{aligned}E \cdot I_B \cdot \theta &= \frac{wl^3}{24} \\ \text{i.e., } E \theta &= \frac{wl^3}{24I_B}\end{aligned}\quad (10)$$

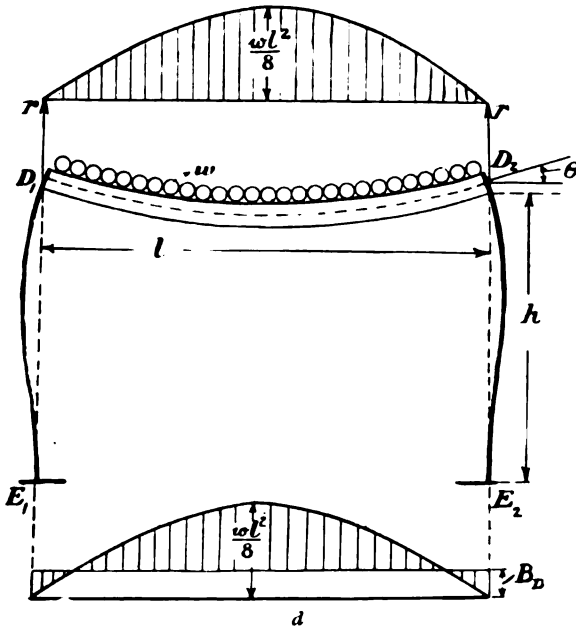


FIG. 5.

column to the point where the stress is required. We will work out our results for the fixed ends of the column.

For columns fixed at the ends (as in Fig. 1) we have, putting this value in (4a),

$$\begin{aligned}B_D &= \frac{4I_c}{h} \cdot \frac{wl^3}{24I_B} \\ &= \frac{wl^3}{6} \cdot \frac{I_c}{h} \cdot \frac{l}{I_B}\end{aligned}\quad (11)$$

Now let $\frac{I_B}{l} \div \frac{I_c}{h}$ be called the stiffness co-efficient and be given the letter s ,

$$B_D \text{ for Case 1} = \frac{wl^3}{6s}\quad (12a)$$

Similarly we should get

$$\text{Case 2 } B_D = \frac{wl^3}{8s}\quad (12b)$$

$$\text{Case 3 } B_D = \frac{wl^3}{12s}\quad (12c)$$

$$\text{Case 4 } B_D = \frac{wl^3}{4s}\quad (12d)$$

Now let the compressive stress due to bending in the column be c_b and let n_c be the distance from the neutral axis of the

$$\text{Now } r_d + r_e = P$$

$$\therefore r_e = \frac{B_D h}{2} \dots \frac{B_D h}{3}$$

$$= \frac{B_D \cdot h}{6}$$

$$= \frac{1}{2} r_d$$

$$\therefore \text{Slope at } E = \frac{1}{2} \text{ slope at } D = \frac{\theta}{2}.$$

Case 3. Other end slopes equally in OPPOSITE direction.—This occurs if the upper floor is loaded on the left of the column and the lower floor equally loaded in the right. In this case the B.M.'s at the two ends must be equal, since the slopes are equal and the curvature is in the same direction. The B.M. diagram will be a rectangle, as shown, and the column will bend to a circular arc.

$$\therefore \text{each imaginary reaction} = \frac{P}{2} = \frac{B_D \cdot h}{2}$$

$$\therefore E \cdot I_c \cdot \theta = B_D \cdot \frac{h}{2} \quad (6)$$

$$\text{i.e., } B_D = B_E = \frac{2E \cdot I_c \cdot \theta}{h} \quad (7)$$

Case 4. Other end slopes equally in SAME direction.—This case is shown in Fig. 4. It would occur if both floors were loaded on left-hand side only.

The slopes at the two ends are the same, so that the B.M.'s will be the same; the curvatures are in opposite directions so that the B.M.'s will be opposite in sign.

Taking moments about *e* we have

$$r_d \cdot h = P \cdot \frac{5h}{6} - \frac{P \cdot h}{6} = \frac{2P \cdot h}{3}$$

$$\therefore r_d = \frac{2P}{3} = 3 \cdot \frac{1}{2} \cdot \frac{B_D \cdot h}{2}$$

$$= \frac{B_D \cdot h}{6}$$

$$\therefore EI\theta = \frac{B_D \cdot h}{6} \quad (8)$$

$$\text{i.e., } B_D = \frac{6EI_c \theta}{h} \quad (9)$$

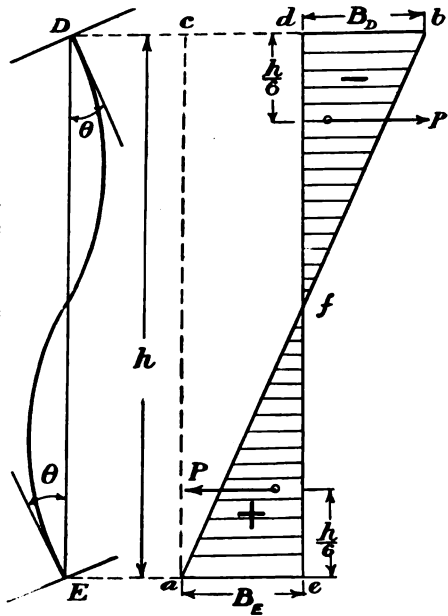


FIG 4

Summary of Results.

Case 1 $B_D = \frac{4EI_c \theta}{h}$

Case 2 $B_D = \frac{3EI_c \theta}{h}$

Case 3 $B_D = \frac{2EI_c \theta}{h}$

Case 4 $B_D = \frac{6EI_c \theta}{h}$

We learn from these four results the very important fact that the less the deflection of column for a given end slope the greater will be the B.M. and therefore the stress, because it is clear that the deflections will be the greatest in Fig. 3, next in Fig. 2, next in Fig. 1, and least in Fig. 4.

Calculation of Slopes.

In a rigid analysis of these problems we should take account of the fact that the rigidity of the columns themselves will affect the slopes which the beams will have, but an allowance for this causes great mathematical complications, except in the simplest cases. We will follow Messrs. Faber and Bowie in neglecting this as a first approximation in some cases; the result will err on the right side, but, as we shall show later, the error will often be considerable unless the beams are very rigid compared with the columns. We will first consider the case of a single span uniformly loaded and carried on columns at the ends. The B.M. diagram for the beam is a parabola as shown in Fig. 5, w being the uniform load per unit length.

$$\begin{aligned}\text{Then } r &= \frac{1}{2} \text{ area parabola} \\ &= \frac{1}{2} \cdot \frac{2}{3} \cdot l \cdot \frac{wl^2}{8} \\ &= \frac{wl^3}{24}\end{aligned}$$

$\therefore$ If E is the same for beam and column and I_B is the moment of inertia of the beam,

$$\begin{aligned}E \cdot I_B \cdot \theta &= \frac{wl^3}{24} \\ \text{i.e., } E \theta &= \frac{wl^3}{24I_B}\end{aligned}\quad (10)$$

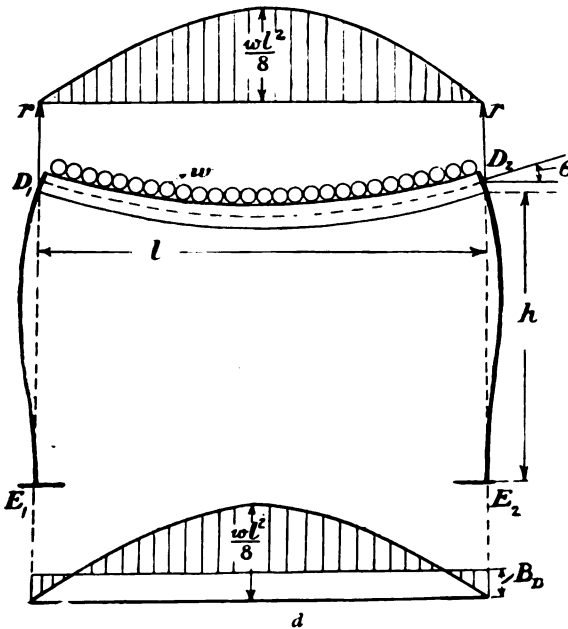


FIG. 5.

For columns fixed at the ends (as in Fig. 1) we have, putting this value in (4a),

$$\begin{aligned}B_D &= \frac{4I_c}{h} \cdot \frac{wl^3}{24I_B} \\ &= \frac{wl^3}{6} \cdot \frac{I_c}{h} \cdot \frac{1}{I_B}\end{aligned}\quad (11)$$

Now let $\frac{I_B}{l} \div \frac{I_c}{h}$ be called the *stiffness co-efficient* and be given the letter s ,

$$B_D \text{ for Case 1} = \frac{wl^3}{6s}\quad (12a)$$

Similarly we should get

$$\text{Case 2 } B_D = \frac{wl^3}{8s}\quad (12b)$$

$$\text{Case 3 } B_D = \frac{wl^3}{12s}\quad (12c)$$

$$\text{Case 4 } B_D = \frac{wl^3}{4s}\quad (12d)$$

Now let the compressive stress due to bending in the column be c_b and let n_c be the distance from the neutral axis of the

column to the point where the stress is required. We will work out our results for the fixed ends of the column.

We then have

$$\begin{aligned} c_b &= \frac{B_E \cdot n_c}{I_c} \\ &= \frac{w \cdot l^2}{6} \cdot \frac{I_c \cdot l}{h \cdot I_B \cdot I_c} \cdot \frac{n_c}{I_c} \\ &= \frac{w \cdot l^2 \cdot n_c \cdot l}{6hI_B} \end{aligned} \quad (13)$$

Again if the beam is designed for a B.M. of $\frac{wl^2}{8}$, and the working stress in the beam is s , and the distance from the neutral axis to the point where its stress is taken is n_b , we then have

$$\frac{wl^2}{8} = \frac{s I_B}{n_b} \quad (14)$$

$$\begin{aligned} \therefore c_b &= \frac{s \cdot I_B}{75 n_b} \cdot \frac{n_c l}{h \cdot I_B} \\ &= \frac{s \cdot n_c \cdot l}{75 n_b \cdot h} \end{aligned} \quad (15)$$

Corresponding results will be obtained for the other cases. As we have explained already, this formula must be used only in the case in which the column is relatively very slight compared with the beam.

Effect of the Stiffness of the Column.

In the present case of a single span with a uniform load we can allow for the effect of the stiffness of the column in the following manner. The B.M. at the tops of the columns will act as reverse B.M.'s on the beam, thus

producing the B.M. diagram shown in the bottom of Fig. 5.

We thus get

$$\begin{aligned} r &= \frac{1}{2} \text{ area of parabola} - \frac{1}{2} \text{ area of rectangle} \\ &= \frac{wl^3}{24} - B_D \cdot \frac{l}{2} \\ E I_B \cdot \theta &= \frac{wl^3}{24} - B_D \cdot \frac{l}{2} \\ \therefore E \theta &= \frac{wl^3}{24 I_B} - \frac{B_D \cdot l}{2 I_B} \end{aligned} \quad (16)$$

For fixed lower end of the column, therefore (using equation 4), we have

$$\begin{aligned} B_D &= \frac{4 I_c}{h} \left(\frac{wl^3}{24 I_B} - \frac{B_D \cdot l}{2 I_B} \right) \\ &= \frac{wl^3}{6} \cdot \frac{I_c l}{I \cdot I_B} - \frac{2 B_D \cdot I_c l}{h \cdot I_B} \\ &= \frac{wl^2}{6 s} - \frac{2 B_D}{s} \end{aligned}$$

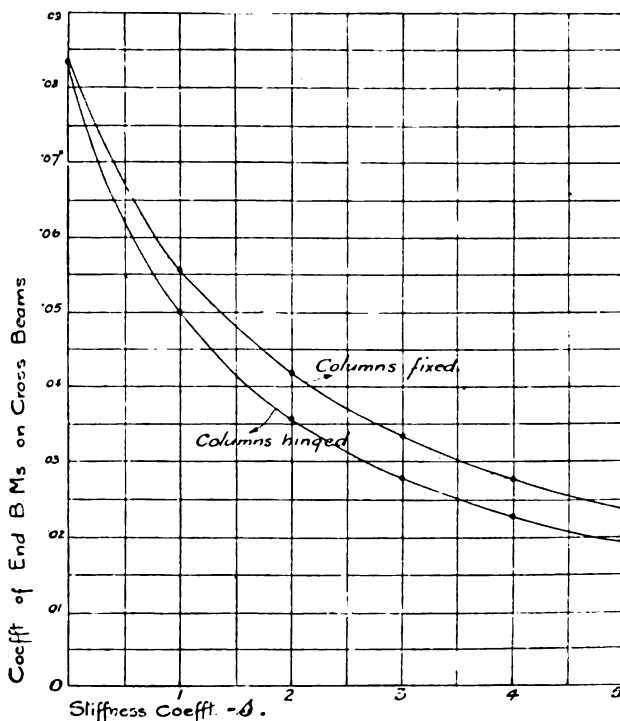


FIG. 6.

$$\begin{aligned}\therefore B_D \left(1 + \frac{2}{s}\right) &= \frac{wl^2}{6s} \\ \text{or } B_D &= \frac{wl^2}{6s \left(1 + \frac{2}{s}\right)} \\ &= \frac{wl^2}{6(s+2)}\end{aligned}$$

By similar reasoning for the other cases we get the following summary of results, which the student should check for himself :—

$$\text{Case 1} \quad B_D = \frac{wl^2}{6(s+2)} \quad (17a)$$

$$\text{Case 2} \quad B_D = \frac{wl^2}{4(2s+3)} \quad (17b)$$

$$\text{Case 3} \quad B_D = \frac{wl^2}{12(s+1)} \quad (17c)$$

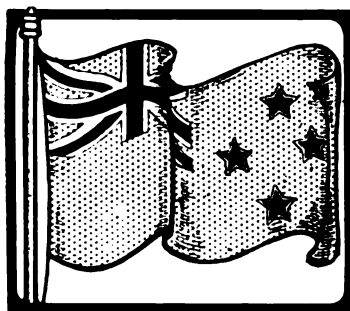
$$\text{Case 4} \quad B_D = \frac{wl^2}{4(s+3)} \quad (17d)$$

If we call $B_D \frac{kEI_c \theta}{h}$ where k is a constant given in the summary of results on p. 13, our result may be expressed in a general expression as follows :—

$$\begin{aligned}\text{From equation (16) } E\theta &= \frac{wl^3}{24I_B} - \frac{B_D \cdot l}{2I_B} \\ \therefore hB_D &= \frac{wl^2}{24I_B} - \frac{B_D \cdot l}{2I_B} \\ \frac{B_D}{k} &= \frac{wl^2}{24} \cdot \frac{I_c}{h \cdot I_B} - \frac{B_D}{2} \cdot \frac{l \cdot I_c}{h \cdot I_B} \\ &= \frac{wl^2}{24} - \frac{B_D}{2s} \\ \therefore B_D \left(\frac{1}{k} + \frac{1}{2s}\right) &= \frac{wl^2}{24s} \\ B_D (2s+k) &= \frac{wl^2}{24s} \\ \therefore B_D &= \frac{k \cdot wl^2}{12(2s+k)} \quad (17)\end{aligned}$$

Fig. 6 shows for various values of s the values of the co-efficients for the end B.M.'s in the beams which are the same as the B.M.'s at the tops of the columns in cases 1 and 2. It will be noted that in each case if $s=0$, i.e. the columns are infinitely stiff compared with the beam, $B=0.83 \, wl^2 \left(\frac{wl^2}{12}\right)$ this being the familiar value for a beam absolutely fixed at the ends. In using these curves, the resulting co-efficients are multiplied by wl^2 to give the B.M. The next case that we will consider is that of a continuous beam of two equal spans rigidly connected to three columns.

(To be concluded.)



REINFORCED CONCRETE POWER HOUSE, NEW ZEALAND.

By H. C. GAYFORD,
Public Works Department, New Zealand.

The following Notes on a Reinforced Concrete Building in New Zealand, in connection with the New Zealand Government's Lake Coleridge Electric Power Plant, may be of interest.—ED.

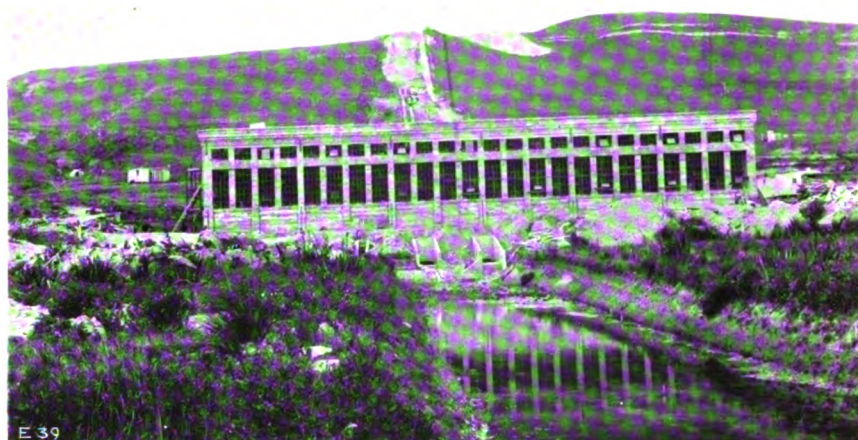
As an illustration of the use of reinforced concrete for building purposes where other materials are difficult to obtain, and as showing the favour with which it is regarded by engineers to the Government, the following brief description is



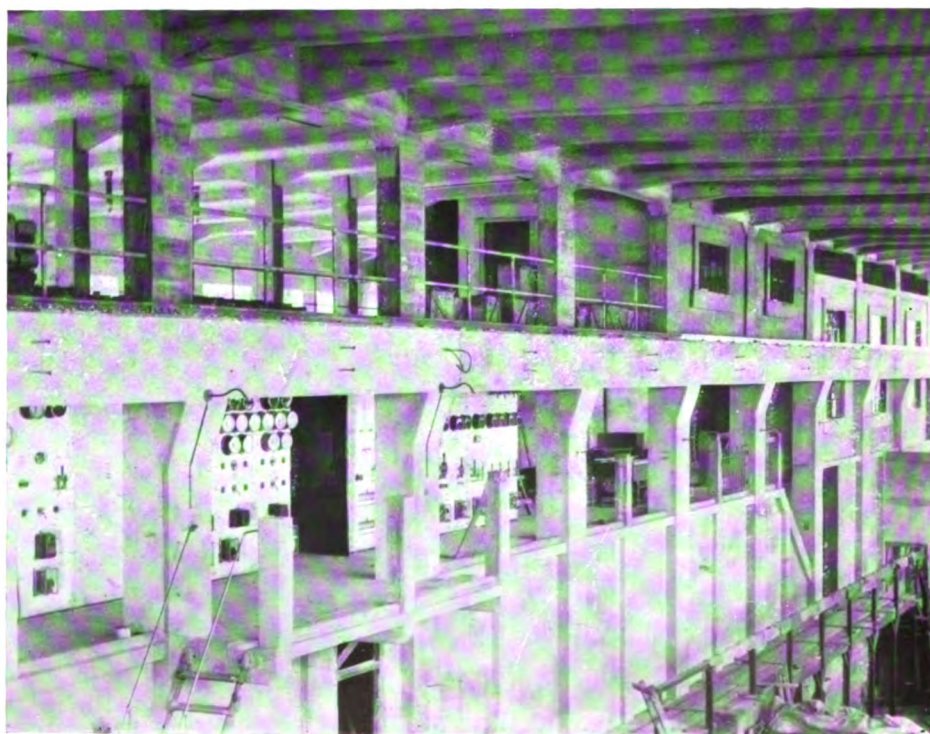
View showing Building in course of construction.
REINFORCED CONCRETE POWER HOUSE, LAKE COLERIDGE, NEW ZEALAND.

given of a power-house built entirely of reinforced concrete at Lake Coleridge in connection with the New Zealand Government's first hydro-electric power installation.

The location of Lake Coleridge is about 70 miles west of Christchurch, and the current generated will be utilised to supply that city with power and light.



View showing Power House and Tail Race, with Pipe Line and Tunnel Outlet in background.



View showing Switchboard Gallery, Upper Gallery, and Reinforced Concrete Crane Runway Beam.
REINFORCED CONCRETE POWER HOUSE, LAKE COLERIDGE, NEW ZEALAND.

The power-house is situated on the banks of the Rakaia River, 1,200 ft. above sea level, and about 2 miles from the lake. The only means of access to the site from the nearest railway station at Coalgate, thirty miles away, is by road, part of which was remetalled by the Government to facilitate transit of material and machinery.



Cross Section.



View looking down Pipe.

REINFORCED CONCRETE POWER HOUSE, LAKE COLERIDGE, NEW ZEALAND.

All material for use in connection with the building (with the exception of shingle and sand) was hauled from Coalgate to the site by traction engines at a cost of 23s. per ton in addition to the cost of railage. This rendered it imperative to utilise local material obtainable on the site as far as possible. Sand and shingle were obtained from the river bed, where it was washed and screened before being carted to the site.

The building, which is entirely of concrete, accommodates the whole of the generating plant, and covers an area of 13,500 sq. ft., its length being 180 ft., width 75 ft., and height from ground to parapet level 35 ft.

For the reinforced concrete a mixture of 1 part cement, 2 parts sand, and 4 parts shingle of $\frac{3}{4}$ in. gauge was used. This mixture, when tested for compression on 4-in. cubes after one month gave an average of 2,400 lb. per square inch.

Concrete for machinery foundations and other dead loads was mixed in the proportions of 1 to $2\frac{1}{2}$ to 5, the shingle being screened to a $2\frac{1}{2}$ -in. gauge.

The steel for reinforcing was of plain round section varying from $\frac{1}{4}$ in. to $1\frac{1}{4}$ in. in diameter, and conformed to the British standard specification for structural steel. When tested in the Government's laboratory in Welling-



View showing Method of Boxing for Parapet.
REINFORCED CONCRETE POWER HOUSE, LAKE COLERIDGE, NEW ZEALAND.

ton a tensile strength of 28 tons per square inch was determined.

Aggregate for the concrete was obtained from the river bed, where it was washed and screened. The method employed for washing was to flume the shingle down a long wooden trough by means of water obtained from a creek at a higher level.

The cement used throughout was of New Zealand manufacture.

A portable bath mixer was used, and was particularly adaptable to this building owing to the small quantity of concrete in the piers, which were spaced over a large area.

The building consists of piers spaced at 15-ft. centres carried up to take the roof beams. These piers are connected at a height of 21 ft. by a band of concrete 2 ft. deep and 12 in. thick extending round the entire building. The lower portion between the piers to a height of 8 ft. above floor level is filled in with 9-in. curtain walls finished with moulded projections inside and out forming the window-sills.

The roof is graded 1 to 35 to shallow gutters formed in the concrete at the back of the parapet wall, and is waterproofed with three-ply reinforced malthoid laid direct on the concrete slab, each layer being placed separately and securely embedded in hot asphalte.

For the wearing surface of all floors, with the exception of those in the office rooms, cement and sand mixed 2 and 1 and finished from a steel float were used.

In the office rooms a patent flooring material was used, which was laid direct on the concrete in two coats. This material is fireproof and very light, thus reducing the load on the beams. A reinforced concrete staircase with a half-landing leads from the ground outside to the office floor level.

No outside plastering was specified, the whole of the exposed surfaces of the concrete being simply finished with two coats of cement wash applied with brushes, after carefully stopping all defects showing in the concrete on the removal of the boxing.



Concrete Entrance Steps.

REINFORCED CONCRETE POWER HOUSE, LAKE COLERIDGE, NEW ZEALAND.

All window openings were fitted with steel sashes and the internal and external reveals plastered up to the frames. The total area of window space is 6,000 sup. ft., constituting a striking feature of the building.

The accompanying cross section will serve to show the general arrangements of which the details are as follows :—

- (a) Main engine-room floor consisting of a 10-in. slab supported on machinery foundations and reinforced with $\frac{3}{4}$ -in. rods at 9-in. centres.
- (b) Basement to accommodate penstock pipes and machinery foundations built up from basement floor to engine-room floor.
- (c) Draught tube chambers.
- (d) 4-ft. 6-in. culvert connecting draught chamber and weir basin.
- (e) Weir basin for measuring the discharge from each turbine.
- (f) Oil switch and busbar chamber.
- (g) Main switchboard operating gallery.
- (h) Upper gallery and battery room.
- (i) Transformer room.
- (j) Three-foot penstock pipes to turbines.
- (k) Reinforced concrete crane beams.
- (l) 20-ton travelling crane.

The total amount of concrete in the building is 2,700 cu. yds., and the percentage of steel to reinforced concrete worked out at about 89 per cent.

At present the only defects showing in the work are small vertical hair cracks in the 9-in. and 4-in. curtain walls caused through expansion and contraction. The cost of the building was £15,800, and the work was carried out by local contractors acting under the supervision of the Assistant Engineer, who was in charge of the whole of the construction works. The designing of the building and the calculation of the stresses were carried out by members of the electrical engineering branch under the supervision of the Chief Electrical Engineer to the Government, Mr. Evan Parry.

RECENT BRITISH PATENTS RELATING TO CONCRETE.

We propose to present at intervals particulars of British Patents issued in connection with concrete and reinforced concrete. The last article appeared in our issue of October, 1914.—ED.

Centering for Concrete Floors, etc.—No. 249/14. J. W. Brockman, "Lynton," Ashbourne Avenue, Blundellsands, Lancashire. Accepted September 24/14.—This invention comprises an improved form of centering for giving the proper contour to floors, ceilings, arches, etc., which can be removed when the work has set solid and then re-used.

The centering consists of a piece of metal (A, Fig. 1), with corrugations (B) in it, the end portions (C) having the crests of the corrugations so folded at b that they become level with the hollows b^1 , and so form flat portions which are bent downwards so as to taper slightly outwards, while the extreme ends are bent laterally outwards to form flanges (D). An inverted trough-shaped centering is thus formed, the upper or unbent portion being corrugated, while the sloping portions (C) and the lateral flanges (D) are flattened.

The crests of the corrugations can be folded as shown either in Fig. 4 or Fig. 5. In either case the metal has three plies or thicknesses at the pleat which strengthens the flattened portion and enables it to be bent downwards, and also facilitates the centering being removed when the work has set.

In use a number of these centerings are placed end to end overlapping one another and in rows spaced apart. A piece of wood or other suitable material (E), Fig. 3, is placed between each row, to which the lateral flanges (D) are nailed or otherwise secured. Concrete (F) is then filled into the spaces between the inverted troughs and on top, and is reinforced in any suitable way, as by bars (G) having wings (H). When the concrete has set, the centering is separated from the pieces of wood (E) between each row, which remain in position, while the centering is removed without injuring the concrete work.

If desired, the flattened portion can be flush with the outward crests (B) of the corrugations, instead of with the inward crests (b^1), and the shape of the centering can be varied.

Fig. 1.

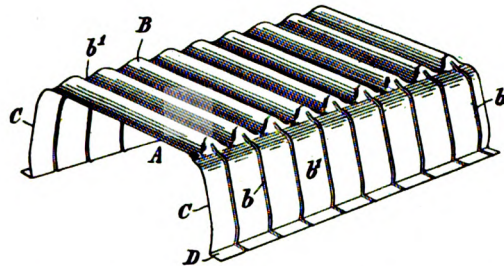


Fig. 3.

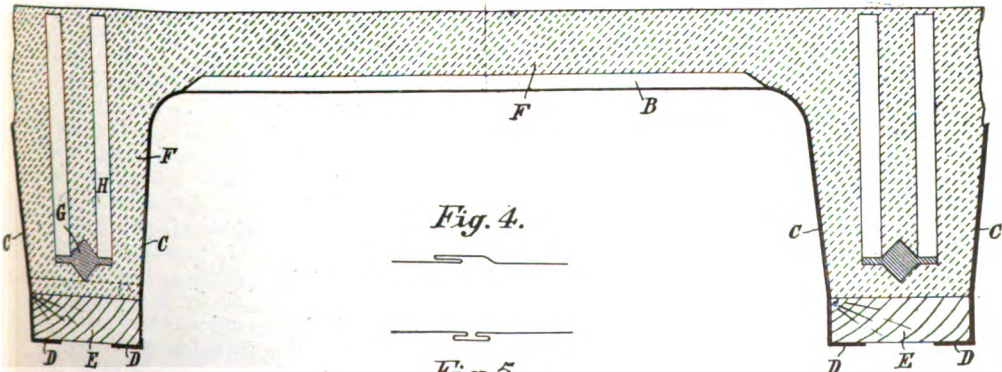


Fig. 4.

Fig. 5.

Reinforced Concrete Walls.—No. 522/14. H. J. Walduck, of Walls, Ltd., Crown Galvanising Works, Fazeley Street, Birmingham. Accepted October 15/14.—According to this invention vertical walls, partitions, and floors of concrete or similar material are reinforced with corrugated expanded metal work, the corrugations having a depth nearly, but not quite, equal to the thickness of the wall or part to be reinforced.

These sheets of expanded metal (a, Fig. 1) are, in the case of a wall, arranged so that the corrugations are in a vertical plane, and uncorrugated sheets (b) are placed on one or both sides, the outer surfaces of the wall being plaster or cement applied to the flat sheets of expanded metal work.

In constructing the wall, the corrugated sheet (a) is placed in position for the full height which the wall is to have, and the plain or uncorrugated sheets (b) are then placed in position and secured on opposite sides of (a) for a short height only. Concrete is then poured into the spaces between the corrugated sheet (a) and the plain sheets (b) to practically the full height of these sheets. Additional plain sheets (b) are then applied to the corrugated sheet (a) so as to increase the height of the plain sheets, and concrete is poured down the spaces so as to fill up the corrugations in the sheet (a).

Boards may be arranged on opposite sides of the metallic reinforcement employed so as to form a temporary mould for the reception of the concrete. When this has hardened, the boards are raised to bring them opposite parts of the reinforcement to which concrete has still to be applied.

Manufacture of Cement.—No. 17,873/13. L. P. Basset, 43 bis rue des Cheneaux, Montmorency, France. Accepted August 7/14.—The present invention has for its object a process for making cement from calcium sulphate, occurring commonly in the form of gypsum, enabling the complete decomposition of the sulphate of calcium

to be obtained, and at the same time avoiding any trace of sulphide remaining in the cement. The characteristic feature of the invention is that the complete decomposition of the sulphate of calcium is obtained by the production of an excess of sulphide of calcium, and this excess of sulphide is then reduced by oxidation, obtained by an excess of air in the furnace in which the operation is carried on.

In the manufacture of 1,000 parts of cement the following proportions may be taken :

Fig. 1

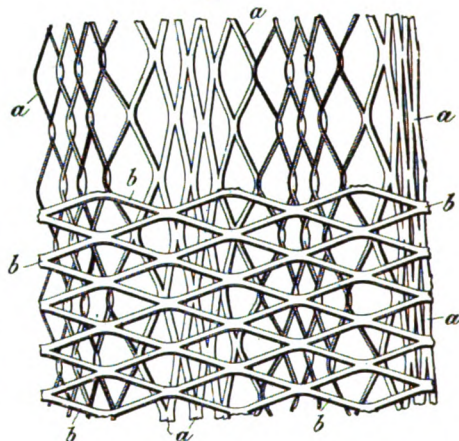


Fig. 2

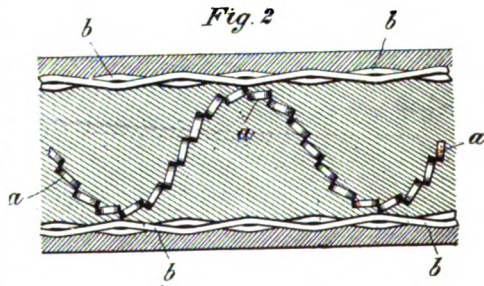
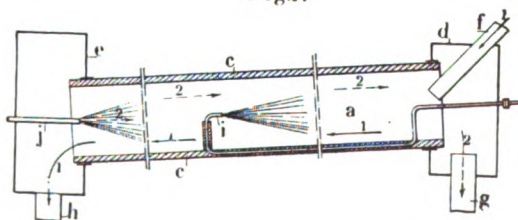


Fig. 1.



Dry sulphate of calcium...	...	...	...	...	1,580 parts.
Dry clay ...	...	...	...	...	350 "
Charcoal (the like material) ...	...	...	...	...	125 "

In order, then, to destroy the excess of sulphide by means of oxygen, as above indicated, the furnace should, in the roasting or second zone, work in an oxidising atmosphere.

A furnace for carrying out the process consists of a rotating cylinder (c) of great length, the ends of which enter two fixed chambers (d and e).

In the first chamber is arranged a shoot (f) conveying the paste formed by the mixture of clay, sulphate of calcium, and a little charcoal or equivalent substance, and a discharging conduit (g) opening into the chimney.

In the second chamber the discharge for the treated products is shown at (h). Carbon monoxide or other reducing agent is injected by a tuyere (i) and air in excess is introduced by a tuyere (j).

In all the zone (a) to the right of the tuyere (i) the atmosphere is reducing; under the action of the carbon monoxide contained in the gases of the furnace, and of the charcoal or the like contained in the paste, a certain quantity of sulphate is transformed into sulphide, and this sulphide is found in excess.

On passing to the left of the tuyere the material encounters an oxidising atmosphere, which causes the decomposition of the sulphide. The roasting of the cement is effected near to the tuyere (j), where the highest temperature occurs. With this new process either sulphurous acid or sulphur can be recovered as by-product.

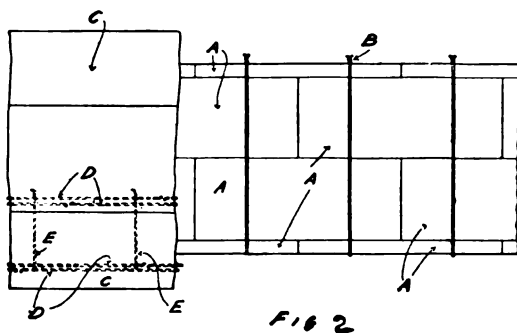
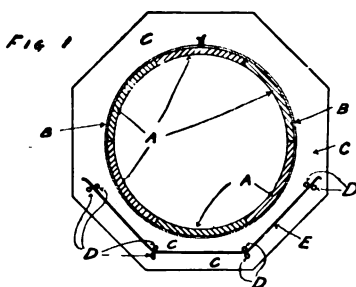
Sewers and Conduits.—No.

17,447/13. *F. Thackeray, 3, Cambrian Terrace, Gwaelodygarth Road, Merthyr Tydfil, Glamorganshire. Accepted January 22/14.*—This invention comprises a construction of sewers, conduits, and culverts by using curved slab linings, which form a permanent part of the sewers, and are bound with wire and cased with concrete, which may, if necessary, be reinforced with iron or steel bars. *Fig 1* shows a cross section of a circular-shaped sewer of this kind, and *Fig. 2* shows a part elevation.

The curved slabs (A), conveniently, but not necessarily, six in number, may be of varied sizes, and may be made of concrete, earthenware, or like materials. Some of the slabs may be made of one material and some of another, and those made of burnt clay may be glazed on the concave surfaces, while those of composition may be reinforced with wires or wire netting imbedded in them during manufacture. These curved slabs support the surrounding concrete during the fresh, soft, wet condition of it as laid *in situ*. The use of special centering tackle is obviated, and when fixed in position the slabs are simply jointed with cement and bound with wire belts (B), and the ends of each piece of wire forming a belt are twisted together.

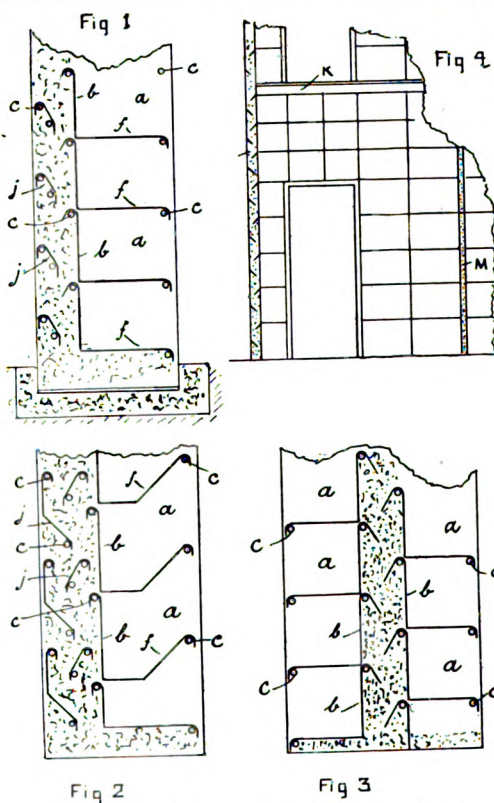
If additional strength is required to support the sewers, etc., on account of the soft nature of the ground, or from other causes, the concrete (C) may be reinforced by iron or steel bars (D) embedded in the lower part of it, and may be further reinforced by bars (E) where required.

The cross-sections, instead of being circular as shown in *Fig. 1*, may be elliptical,



oval, or egg-shaped. During and after construction holes may be formed in the slabs and concrete for junction pipes, vent pipes, etc., or junction blocks may be built in for this purpose.

Reinforced Concrete Warehouses.—No. 28,584/13. P. T. J. Estler, 15, Dowgate Hill, Cannon Street, London. Accepted August 27/14.—This invention relates to the construction of walls of reinforced concrete buildings designed for warehouses, store rooms, libraries, etc., and consists in constructing them in such a manner that the walls themselves form shelves, bins, or hoppers for the reception of goods.



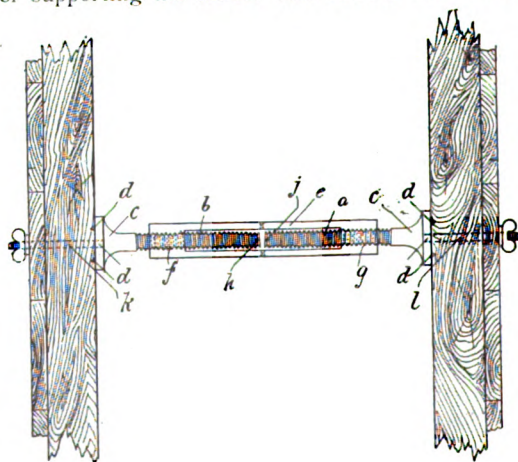
Figs. 1 and 2 show sections of outer walls of a building, Fig. 3 shows a section of a partition wall, and Fig. 4 a section of part of a building.

Sheets of iron or steel (a) are strung together on rods (c). The shelves (f) have their backs (b) elongated and sloped downwards over the hooks (c), and the interspaces are filled with concrete, thus making a wall combined with shelving. In Figs. 1 and 2, which are sections of outer walls, additional bolts (c) and bent plates (j) are employed. All these bent plates can, when required, be perforated for the better securing of the concrete.

The outer walls, and, in some cases, the inner, can be built in sections or nests, the interspace being filled with concrete, as shown at (M), Fig. 4. The flooring between room and room can be treated in the same way, as shown at (K).

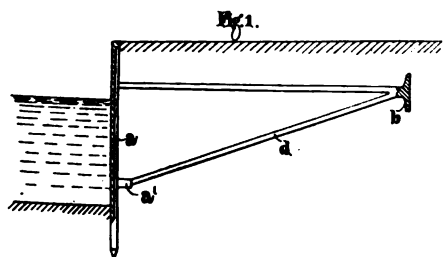
Struts for Excavations and Concrete Shuttering.—No. 449/14. F. W. Wale. Tunbridge, Wolseley Road, Wealdstone, Middlesex. Accepted August 7/14.—This invention relates to improved struts for supporting the timber that retains the soil in place in excavation work, or maintains in position the moulding boards used when making concrete walls.

The strut comprises a pair of abutment members (a and b), one of which has a right-hand and the other a left-hand thread. Each has a head (c) at its outer end provided with sharp projections (d) to engage the structure to which the strut is applied. The members a and b are held together by an elongated nut (e), which has internally-screwed portions (f and g) to engage the right- and left-hand screw members a and b, and on the inner ends of these screw members pins or projections (h and j) are situated to prevent the normal complete separation of the parts.



The elongated nut (*e*) may have the central portion cut away, so that a tool may be inserted to turn it and so effect adjustment of the screws (*a* and *b*). In modified constructions it may be of tubular form, having an aperture into which a tool may be inserted, or it may be of angular form to be turned by a spanner. When the strut is required for forms, shuttering or bordering for supporting concrete in its wet condition, additional support is provided by means of bars (*k* and *l*) secured to the tie in any suitable manner, as, for instance, by screws in the heads (*c*), and the bars are then passed through the moulding boards and secured in position by means of ordinary wing nuts.

Concrete Subaqueous Structures.—No. 22,812/13. *E. Coignet, 20, rue de Londres, Paris.* Accepted August 7/14.—According to this invention main elements, such as



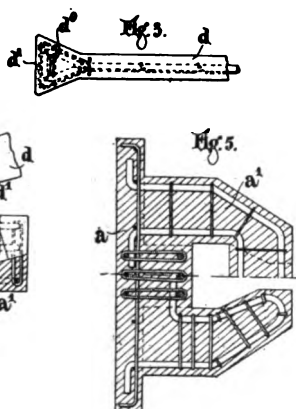
piles or sheet piles (*a*, Fig. 1), and supplementary members, such as ties, stays, or like members (*d*), are provided each with one part of a dovetail or like joint, the arrangement being such that the stay, tie, or like member is coupled to the element by the interlocking of the joint parts.

The invention is described with reference to a quay wall formed of sheet piles (*a*), an anchor slab (*b*), and two super-posed series of ties.

Fig. 3 shows an enlarged view of one of the ties (*d*), and Figs. 4 and 5 are respectively a second elevation and an inverted sectional plan (in two planes) of that part of a sheet pile to which a corresponding tie is to be joined.

The ties (*d*) to be used are so formed that each comprises at the end to be assembled under water a head (*d1*) which is of dovetail shape. The reinforcement is suitably designed to correspond to this shape, but is not allowed to project beyond the surface of the head.

The sheet piles (*a*) adapted to engage with the ties (*d*) are formed with projecting members (*a1*), provided with recesses which in plan have the same shape as the heads (*d1*) of the corresponding ties. The members (*a1*) may be suitably reinforced, the re-

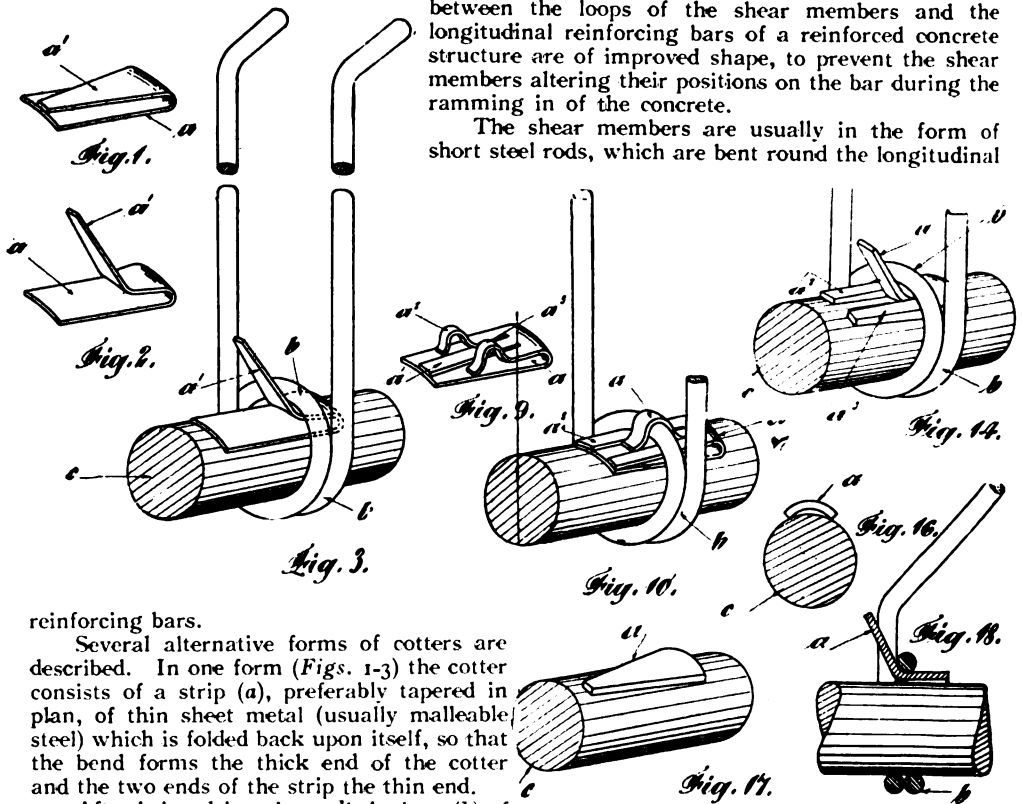


inforcements being arranged so that the surface of the point is uninterrupted by projections.

The various parts to be used are erected in the usual manner, exception being made with regard to the setting in place of each tie (*d*) and the positioning of the head (*d1*) thereof. The tie heads (*d1*) may be formed during manufacture with projecting rings (*d0*), so that ropes may be attached for lowering the ties into position; the rings also serve to identify the face of the tie which is to be arranged uppermost.

In assembling the parts, each tie (*d*) is lowered so that its head (*d1*) may slide vertically along the sheet piles with which it is to be assembled. The head of each tie thus lowered is forced into the recess of the member (*a1*).

Shear Reinforcements.—No. 15,309/14. *The British Reinforced Concrete Engineering Co., Ltd., 82, Victoria Street, Westminster, London, and A. G. Bullivant.* Accepted October 22/14. According to this invention the cotters which are driven in



between the loops of the shear members and the longitudinal reinforcing bars of a reinforced concrete structure are of improved shape, to prevent the shear members altering their positions on the bar during the raming in of the concrete.

The shear members are usually in the form of short steel rods, which are bent round the longitudinal

reinforcing bars.

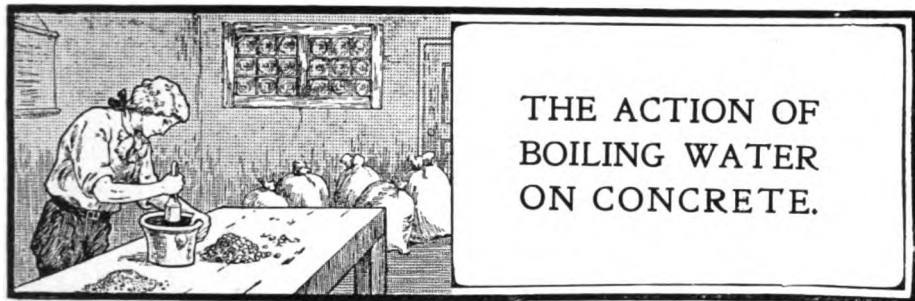
Several alternative forms of cotters are described. In one form (Figs. 1-3) the cotter consists of a strip (a), preferably tapered in plan, of thin sheet metal (usually malleable steel) which is folded back upon itself, so that the bend forms the thick end of the cotter and the two ends of the strip the thin end.

After being driven in until the loop (b) of the shear member is held firmly to the bar (c), the upper thickness of its narrower end is bent upwards and caused to lie immediately in front of and against the loop (b), thus preventing slackening of the cotter with subsequent operations.

Figs. 9 and 10 show different methods of application of similar bent-back cotters.

The cotter may also be made of a slitted solid wedge-shaped piece of metal instead of a thin piece folded back upon itself, as shown in Fig. 14, the centre or the two outer tongues being bent upwards.

In a further modification the cotter is formed of a piece of bendable metal tapered in plan, but of even thickness from end to end, which at its narrower end fits the bar, but at its wider end is bent to afford a "three-point" contact with the bar and loop (Figs. 16-18).



By PROFESSOR E. R. MATTHEWS, A.M.Inst.C.E., F.R.S.E.

(Chadwick Professor of Municipal Engineering in the University of London).

The action of boiling water on Concrete is a question of considerable importance in connection with certain works, and the following short article will doubtless be of interest to engineers.—ED.

TANKS constructed of concrete or reinforced concrete are often made for the purpose of containing hot water, especially in connection with chemical works, and the effect upon the concrete of immersion or partial immersion in water at a temperature of boiling point is one which should be seriously considered. While many experiments have been made to ascertain the action of high and low temperatures upon concrete—such, for example, as the effect of frost on concrete, and the result of subjecting concrete to excessively high temperatures (dry), there appears to have been no thorough investigation with a view of ascertaining whether hot water strengthens or weakens concrete.

The author's attention was drawn to this in a practical way at Bridlington. While serving as Borough Engineer of that town he constructed (about four years ago) at the upper waterworks, and adjoining the boiler house of the new pumping station, a concrete tank for the purpose of receiving the waste hot water from the boilers. This was of plain concrete, and the dimensions were approximately, depth 6 ft., length 4 ft., width 3 ft.; thickness of walls about 9 in. at top and 14 in. at bottom; thickness of concrete floor 6 in. The tank withstood the ill effects (if any) of the boiling water well, but the cement in the joints of the sanitary pipe drain which he connected to this tank, and through which drain the hot water from the tank discharges, showed, when opened out and inspected by him about four months ago, and for some unaccountable reason, that the cement had deteriorated, and that several of the joints were in consequence leaky. A one-to-one (cement and sand) mixture was used for these joints, and the work the author was assured was done well, although the pipes were not subjected to the water test. So leaky, however, were the joints that the drain had to be taken up and relaid. It was laid through chalk. The cement certainly showed that it had deteriorated.

The author took the matter up with the firm who supplied the cement, and was convinced that it was not the cement that was at fault. Thinking that probably the action of boiling water on cement was deleterious, he asked Messrs. G. and T. Earle, cement manufacturers of Hull, who had previously on several occasions carried out tests for him, if they would undertake to investigate this matter of the action of hot water on concrete. They kindly agreed

Concrete Boiling Tests.

Carried out for Prof. E. R. Matthews.

The figures represent actual crushings in tons of the 6 in. Cubes.

TABLE No. 1.

Concrete.	Air Tests.					Water Tests.				Boiling Tests.		
	8 Days.	14 Days.	1 Month.	3 Months.	6 Months.	12 Months.	1 Month.	3 Months.	6 Months.	12 Months.	24 hrs. in air, 7 days boiling.	7 days in air, 7 days boiling.
Trent Gravel, 3½ parts	50 51 52	56 51 52	72 73	80 88	—	—	79	110	—	—	110 95 90	92 88 89
Trent Sand, 1½ parts	63 58 57	80 77	82 78	95 105	—	—	70.5	118	—	—	102 100 100	108 105 84
Cement, 1 part	52 53 48	58 58 58	75 75	85 78	—	—	76	106	—	—	89 97 96	106 99 105
Broken Bricks, 6 parts	34 34 36	39 39 38	45 46	64 66	—	—	44	59	—	—	51 53 58	57 59 62
Trent Sand, 3 parts	34 36 32	37 37 39	45 43	54 58	—	—	39	56	—	—	47 46 48	54 58 56
Cement, 1 part	34 37 37	41 38 39	47 42	49 49	—	—	50	52	—	—	53 47 44	53 56 56
Whinstone, 4 parts	53 55 49	58 58 58	74 69	80 84	—	—	79	118	—	—	80 77 68	95 96 96
Trent Sand, 2 parts	43 41 44	55 54 57	64 58	77 81	—	—	77	91	—	—	79 73 72	80 85 82
Cement, 1 part	40 42 41	47 51 51	55 65	65 69	—	—	75	86	—	—	65 60 62	81 84 68

TABLE No. 2.

Trent Gravel, 3½ parts	208	253	303	360	—	—	309	445	—	—	390	384
Trent Sand, 1½ parts	—	—	—	—	—	—	—	—	—	—	—	—
Cement, 1 part	—	—	—	—	—	—	—	—	—	—	—	—
Broken Bricks, 6 parts	—	—	—	—	—	—	—	—	—	—	—	—
Trent Sand, 3 parts	139	154	179	227	—	—	177	236	—	—	196	227
Cement, 1 part	—	—	—	—	—	—	—	—	—	—	—	—
Whinstone, 4 parts	—	—	—	—	—	—	—	—	—	—	—	—
Trent Sand, 2 parts	—	—	—	—	—	—	—	—	—	—	—	—
Cement, 1 part	181	219	259	304	—	—	308	393	—	—	279	341

Average results in tons per sq. ft.

to do so, and the accompanying tabulated statements set forth the results of their investigations, and are extremely satisfactory and assuring. The tests clearly show that instead of weakening concrete, hot water, up to a period of three months at any rate (I cannot say what takes place beyond that period, but there is every reason to believe that if a detrimental effect was likely to occur, it would have shown itself by the end of three months) increases in compressive strength Portland cement concrete.

The experiments were carried out in three different series and at different times.

Table I shows the actual results of the crushing of the 6 in. cubes.

Table II. gives the average results worked out in tons per square foot.

Conclusion.—Allowing for slight irregularities in the results, the tests show that, if concrete is allowed to harden for seven days, and is then boiled for seven days (which is certainly a severe test), it shows a compressive strength at the end of the fourteen days, which is only attained at three months when the concrete is left in air; when the concrete is immersed in water the equivalent strength would appear to be attained sooner, possibly some time between one and three months.

It must not be assumed that these results are final, although they certainly would justify us in concluding that the idea that hot-water weakens concrete is an erroneous one.

Testing Laboratory Notes on the Cement used.

Cubes and Briquettes filled August 7th, 1914.

TENSILE STRAIN.

Moulds filled with thumb pressure, 1 day in air, 6 days in water.	Neat cement, 1 in. section	7 days.	28 days.
		795 lbs. ..	760 lbs.
Moulds filled by hand with small brass rammer, 1 day in air, 6 days in water.	Neat cement, 1 in. section	825 lbs. ..	930 lbs.
Moulds filled using Standard Bohme hammer, 1 day in air, 6 days in water.	3 standard sand, 1 cement, 1 in. section	410 lbs. ..	510 lbs.
Moulds filled by hand with small brass rammer, 1 day in air, 6 days in water.	3 standard sand, 1 cement, 1 in. section	500 lbs. ..	600 lbs.

COMPRESSION STRAIN.

2 1/2 in. cube mould filled by hand with small brass rammer, 1 day in air, 6 days in water	Neat cement	25.8 tons, 179 tons on sq. ft.	32.8 tons, 609 tons on sq. ft.
1 in. cube mould filled, using Kleb hammer, 1 day in air, 6 days in water.	3 standard sand, 1 cement	10.8 tons, 200 tons on sq. ft.	15.2 tons, 282 tons on sq. ft.

The lead sealed "Pelican" brand of cement used is ground to the following fineness: 2 per cent. residue on 180 sieve—i.e., 32,400 holes to the sq. in.



Concrete Seat.

CONCRETE FOR ORNAMENTAL GARDEN WORK.

For the following illustrations and particulars of concrete work suitable for garden decoration, we are indebted to the "Concrete-Cement Age" of New York.—ED.

CONCRETE is undoubtedly becoming more and more popular for decorative outdoor work, and the illustrations we are able to reproduce give an excellent idea of the many uses to which it can be put. All the work illustrated in this article has been carried out on an estate in Philadelphia under the direction of Mr. Adolph Schilling, and we are able to give a short account of his methods on a later page.

The illustration in our heading is from a photograph of a concrete seat in front of the pergola, the griffins at each end being specially well modelled. The various flower urns and flower boxes need no description, but are interesting as showing what can be done in this way.

The columns in the pergola are of massive cast stone, and a second pergola which we illustrate has four cast-stone statues, emblematic of Spring, Summer, Autumn, Winter. The outlookers and rafters of this pergola are cedar logs, which rest directly on the statues. In another view we show three of these statues in better detail.

The plain work of all these was cast in plaster moulds and the ornamental



Concrete Flower Vases.

CONCRETE FOR ORNAMENTAL GARDEN WORK.

undercut work in gelatine moulds. After casting, at the age of 10 days, the pieces were immersed in large tanks holding an acid solution, until the outer cement coating was removed. This treatment left a fine-grain texture similar to rubbed stone, and shows the aggregate to best advantage. The general colour tone is a warm grey and ivory, resembling in every respect the marble ornaments of the old gardens of Europe.

The following is an extract from Mr. Schilling's paper on his methods in ornamental concrete work :—

Colour Effects.—To produce colour effects we may use the grey and



Concrete Flower Urn.

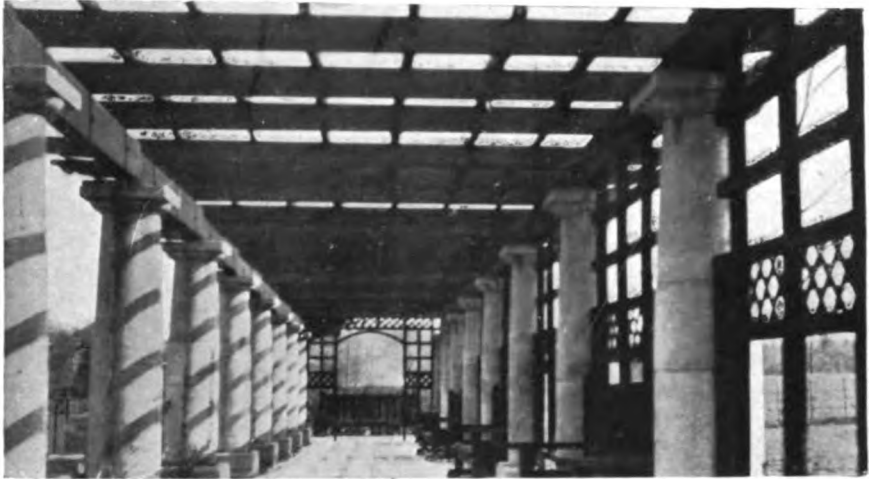
CONCRETE FOR ORNAMENTAL GARDEN WORK.

white Portland cements, either by themselves or mixed in certain proportion, adding to them suitable pigments. But in many cases the natural-coloured aggregates, sand, silica, pebble grits, marble and granite, will give excellent and more uniform results. The importance of mixing the pigment thoroughly with the cement, before adding the aggregates, should be appreciated by anybody attempting to make concrete in colours, and there should be great care to avoid undue weakening of the cement.

As a very simple method to test the proper amalgamation of the pigment with the cement, take a handful of the mixture and press it under a sheet of stiff paper. This will produce an even surface of the material, and as long as

this surface does not show absolute uniformity in colour the mixing is incomplete.

The absorptive qualities of concrete during its stage of curing and seasoning offer opportunities for colouring concrete products by capillary action. By



Interior of Pergola with Concrete Columns



Concrete Pergola—View from the Garden.
CONCRETE FOR ORNAMENTAL GARDEN WORK.

this method the colour is deposited in the pores of the surface, amalgamating with the concrete in a permanent unit. The possibilities of this treatment are unlimited, but individual knowledge of colouring values, and judgment so as not to impair the strength requirements of concrete, are essential for success.

Penetration Method of Colouring.—Colouring solution can be made to penetrate the surface of concrete six inches or more, if the object is placed in



Concrete Flower Urns.

CONCRETE FOR ORNAMENTAL GARDEN WORK.

the solution in a very green state. It is rarely necessary to penetrate more than $\frac{1}{32}$ in. to $\frac{1}{8}$ in.; this thoroughly fills all pores, gives the desired colour effect, and is less expensive. Every atom of colouring matter absorbed by

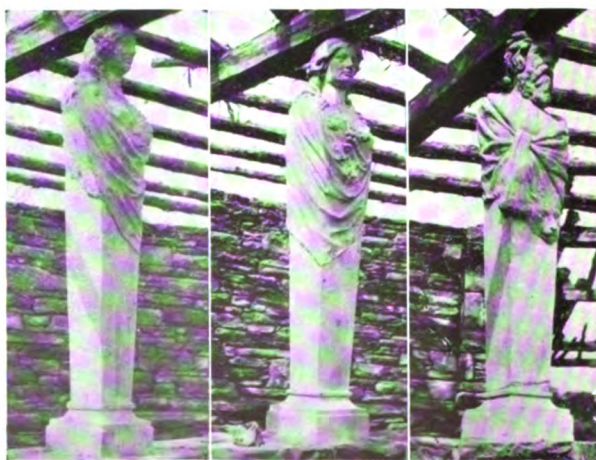


General View of Pergola and Seat.
CONCRETE FOR ORNAMENTAL GARDEN WORK.

the concrete reduces the strength of the solution; and as some of the colouring matter used is quite expensive, good judgment in allowing only the necessary absorption of colouring matter is advisable from an economic standpoint.

Aniline colours and the sulphates of copper and iron are the most suitable to make solutions in which to colour concrete by the capillary method. The concrete to be coloured can be treated after it is several days old. Concrete products with strength requirement should not be subjected to the colouring bath until the concrete has attained its required strength, as the filling of the pores in the concrete stops the action of its curing by the usual methods.

Colouring by absorption is effective on surfaces of concrete after it comes out of the mould, or after being treated with acid or tools. Surfaces that have been coloured by absorbing mineral or metallic colours become waterproof, and the action of the weather on the metallic colours is the same as on real metals, increasing the beauty of colouring by the usual oxidation noticed on bronze or copper. Surfaces of concrete treated by this method become so hard and dense that they will take a uniform dull or high gloss polish. I have treated such surfaces in the same manner as marble, granite, and metal, under polishing or buffing machines.



Concrete Figures in Pergola—showing details.
CONCRETE FOR ORNAMENTAL GARDEN WORK.

Products made by these methods, such as flower pots, vases, and boxes, will hold water after the second day of casting, and become so hard that when struck with a hammer they sound like a metal bell. I do not think water-proofing compounds are essential in obtaining this result, but consider the proper amount of water and thorough grading as all important. I have obtained excellent two and three colour effects by painting certain parts of objects before subjecting them to the colouring bath. The parts so coloured would not be affected by the colour in the bath.

The artistic possibilities of such treatment are limited only by the colour sense and taste employed by the craftsman. By using certain non-absorptive



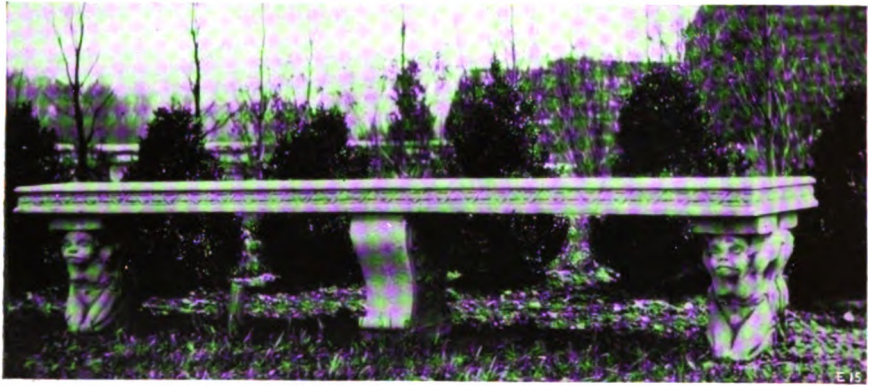
Pair of Concrete Flower Boxes.
CONCRETE FOR ORNAMENTAL GARDEN WORK.

aggregates, their natural colour can be retained, while the absorptive parts will assume the desired colour. In this treatment certain precautions must be taken not to use certain acids in washing before immersion in the colour bath, as the chemical action of the acids is likely to counteract the colour value of the bath.

I find that concrete of proper age can be treated just like any natural stone, using the same tools and machinery to dress its surfaces, and it is my strong conviction that the success of concrete stone for building purposes rests with a close affiliation of the stone-caster and the stone-cutter. In this way alone shall we be able to give concrete proper texture and the necessary qualities of dimension stone, so essential with the architect and builder.

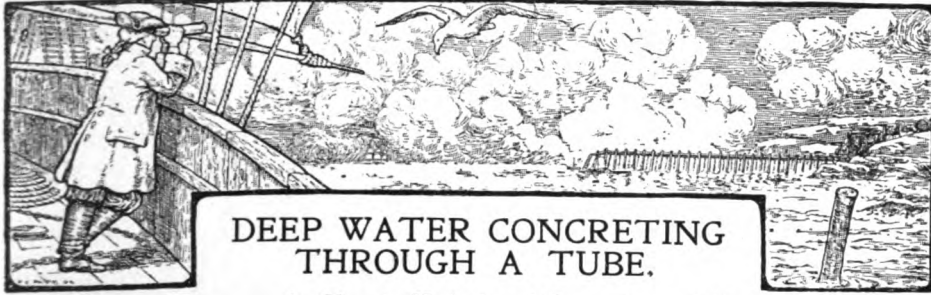
Acid Baths.—For acid baths, tanks of sufficient size should be constructed in a concrete shop, and the soaking of concrete surfaces in acid will not only result in a great saving of acid, but produce a class of work that cannot be obtained with the scrubbing brush. This treatment preserves the edges and details of design, and makes the surface uniform. Any of the hard spots not

sufficiently affected by the acid bath can be treated separately after the article has been flushed with clear water. Care must be taken that aggregates of the surfaces are of nearly uniform hardness, or the acid will eat the soft portions out before the harder particles have been cleansed of the cement coating. I have had some very fine work spoiled when, to obtain a certain fine effect, I mixed black marble (a limestone) with crushed granite. The acid bath left only the spaces where the black marble had been, while the granite showed a very fine texture and natural colour.



Concrete Garden Seat.

CONCRETE FOR ORNAMENTAL GARDEN WORK.



Notes on the Tremie Method and Suggestions of Various Expedients to Avoid the Difficulties Likely to be Encountered.

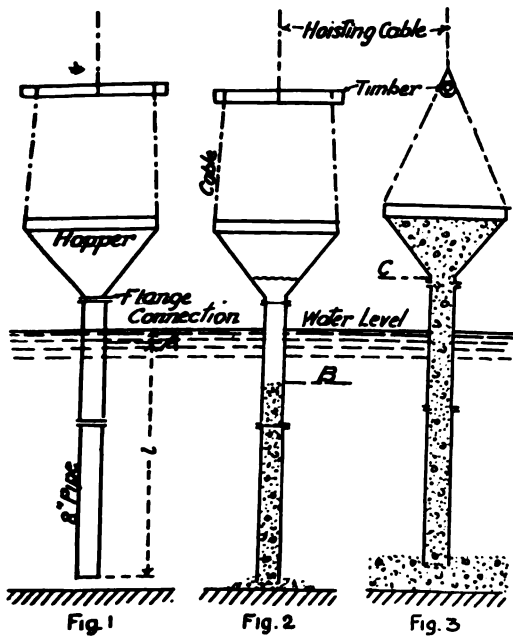
By E. B. Van De GREYN.

We publish below (slightly abbreviated) an article which has appeared in the "Engineering Record" on Deep Water Concreting, thinking the matter may be of interest to some of our readers.—ED.

SOME time ago in a short article on the method of starting deep-water concreting through a tube it was stated that "as is well known, unless the tube is sealed in some way the cement will wash out during the downward passage and leave only a pile of stone and sand. This waste of concrete may in the case of a 12-in. tube 30 ft. long amount to more than 1 cu. yd."

It is plain that if the tube is filled by dumping little batches (such as wheelbarrow loads) the concrete in each small batch will be washed until the water in the tube is absorbed or forced out at the top. If, however, a large batch

of concrete, sufficient to fill a considerable length of the pipe, say 30 ft. or 40 ft., is dumped at one time into the hopper I would not expect the concrete to be washed as stated above—that is, if the procedure of charging outlined hereafter is followed.



TREMIE EMPTY, SEALED AND READY TO BE SHIPPED.

Experimental Tremie. — A laboratory experiment with a miniature tremie was tried by the writer. The tremie consisted of a glass tube 1 in. in inside diameter and 12 in. long and a tin funnel cut to fit the top. A china plate was placed in the bottom of a water bucket partly filled with water. The glass tube was clamped to a standard with its lower end 5 in. below the water surface and $\frac{1}{8}$ in. above the china plate. A batch of mortar was made up of 1 part of cement and 2 parts of standard Ottawa sand, the mortar

being made quite wet so as to flow freely through the tube. The batch of mortar was dropped into the hopper several times with the same results. In each case the water was driven out of the tube ahead of the mortar and the mortar spread out over the plate and filled the lower part of the tube. In each case there was no washing out of cement. The mortar extended about 5 in. up in the tube. A depth of water of 5 in. is small compared with 40 ft. or 50 ft., but the experiment shows what might be expected.

Sealing the Pipe.—There is no doubt that preliminary sealing of the pipe in some way is the safe way to proceed to charge the pipe for the first time. Under certain conditions of concreting under water I would consider that a layer of shavings and sawdust with a layer of cement on top should be placed in the pipe before starting the concrete, but in some instances the plug might be dispensed with. For example, consider the conditions on one job under the writer's charge. A large open caisson had been sunk to a depth of about 50 ft. below water level in alluvial soil, the caisson had been dredged out and piles driven within, and finally the soil that was forced up during driving had been removed. The excavation was carried a little lower than depth required on the plans. The concrete placed immediately on the bottom would mix with the mud and form a base for the remaining concrete required on the plans, and due to the large mass of concrete to be placed in the caisson it was not necessary often to change the position of the tremie in a horizontal direction. Under such conditions I believe charging the tremie according to the method outlined below was ample protection against washing out of the cement.

The following method of deep-water concreting has been used under the charge of the writer: Inspection after pumping out the water has always revealed the top concrete to be first class. The method and equipment, together with precautions to be observed, are submitted with the hope that some of the ideas may be of benefit to those who may find themselves for the first time in charge of such work.

A tremie is shown in *Figs. 1, 2 and 3* made up of sections of water pipe with bolted flange connections and a metal hopper large enough to hold one batch of concrete. The hopper is bolted to the top section of the pipe. All connections are made watertight by means of gaskets. An 8-in. pipe has proved practicable and has the advantage over larger pipe in being lighter.

On one job the tremie was raised and lowered with the hammer line of the pile driver stationed on falsework over the caisson. This proved very satisfactory, as the up-and-down movement of the tremie was under perfect control, and such control is important in tremie work. On another job the tremie was suspended from the load line of a barge derrick. This proved serviceable, but not so satisfactory as the pile driver because of swaying of the tremie due to wave action on the barge and due to tipping of the barge at the time of raising the tremie.

In charging the tremie a large batch capable of filling a considerable length of the pipe should be discharged into the hopper at one time. Where the contractor is not equipped with a derrick a contractor's cart holding a large enough batch could be used, or if wheelbarrows are used a hopper with a trap

door would serve the purpose of discharging a large batch suddenly into the tremie hopper. After the pipe is once sealed the concrete from then on can be dumped into tremies in large or small batches.

Concrete should be Mushy.—Concrete deposited in deep water through a pipe should be of a mushy consistency in order that it may flow easily after leaving the end of the pipe under the surface of concrete just previously deposited; it will then spread more evenly after leaving the pipe, thus avoiding the formation of hillocks and making it unnecessary to change the position of the tremie in a horizontal direction as frequently as would be the case if drier concrete were used. Also, it will not choke the pipe. A little experimenting at the outset will soon indicate how much water should be used in each batch. After the proper amount is determined it should be regulated by a gauge so as not to vary, consideration being given, of course, to appreciable changes in the amount of moisture in the aggregate.

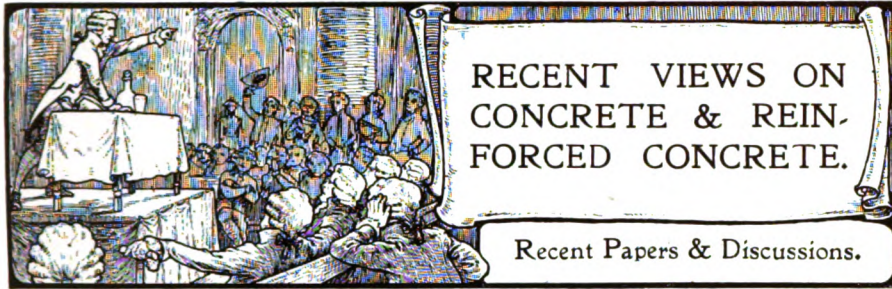
Mushy concrete deposited in water should have an excess of cement. There is a loss of cement due to the excess water in the concrete finding its way out, carrying a portion of the cement, which is found in the laitance on top. A 1 : 2 : 3 mix is recommended.

Operation.—In the illustration *Fig. 1* shows the tremie ready to receive the first batch, the bottom of the pipe being raised from 6 in. to 12 in. above the bottom of the excavation, so that the water may escape as the concrete comes down the pipe. It is well to land the pipe on the bottom and then raise it, so that one is assured that it is above the bottom. Assume that one batch would fill the pipe for a length l —that is, when the concrete begins to emerge from the end of the pipe the top surface is at level A . If the pipe is to have a plug of planer shavings and cement they should be in place before dumping concrete into the pipe. Charging the pipe is the most difficult part of deep-water concreting through a tube, hence it should be supervised by some competent person. If the pipe is dropped before concrete emerges from the lower end the water in the lower part of the pipe cannot escape and it boils up through the concrete, washing out the cement. Whether a new charge or the absorption of this water by additional drier concrete is better must be determined by judgment, and the amount of such water should decide which will result in the lesser waste. If the dropping of the pipe in making the first charge is delayed too long all the concrete escapes, which means that one batch is partly lost and the operation must be repeated. After the batch is dumped into the hopper, when the surface of the concrete drops to level A , the tremie should be dropped. By the time the pipe is dropped to the bottom the surface of the concrete will be below level A , as indicated in *Fig. 2*, and the tremie will be sealed.

After the pipe is sealed with concrete as shown in *Fig. 2* the remainder of the pipe and hopper can be filled as shown in *Fig. 3*. Next the tremie should be raised slowly until the concrete flows downward. It is not safe to raise the pipe continuously until the concrete begins to flow, because by the time the "stop" signal is obeyed the pipe will have been raised too far and there will be a sudden rush of concrete through the pipe, with accompanying danger that the pipe will not be lowered quickly enough to prevent all concrete from

leaving the pipe and leaving it unsealed. Therefore the tremie should be raised a few inches and held, and then raised again a few inches and held, repeating until the correct vertical position is found where downward movement will begin; the concrete will then slowly lower in the pipe and will emerge from the bottom under right conditions. All this can generally be done in short time and will cause no delay at the mixer, especially after the men become familiar with the work. When the surface of the concrete lowers to some level, say C, at which it is desired to stop the flow, the tremie is lowered and the flow ceases. There is some definite point at which the pressure on the end of the pipe due to head of water and concrete outside of the pipe plus the resistance of the concrete to move through the pipe is equal to the weight of the concrete. It is to locate this point that the tremie should be raised a little at a time as described in the foregoing. Concrete can thus continue to be placed under water without passing through water, avoiding all wash, emerging as it does from the pipe under the surface of the concrete just previously placed. It is well never to let the surface of the concrete go below the bottom of the hopper, since it is then always in sight; if it is allowed to go below this the next batch confines the air in the tube, most of the air bubbling up through the concrete.

Precautions.— See that all connections are watertight. Lash the several sections of the pipe together with cable so as to be able to save them if they break loose. Examine the connections whenever possible to see that they are not broken. If a connection were broken and the break large enough to let concrete through great damage would result by concrete flowing through the break and dropping through the water, causing segregation of the constituent materials. After starting concreting keep it up continuously until completion and regulate the moving of the tremie horizontally so as to keep the laitance on top where it can be removed later. After the water is pumped out, clean the surface of the concrete thoroughly, so as to remove all laitance and furnish a good surface on which to start other concrete work.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE CONCRETE INSTITUTE.

SHEAR AND PROBLEMS ARISING THEREFROM.

By H. KEMPTON DYSON.

A most interesting Paper was presented at the Fifty-second Ordinary General Meeting of the Concrete Institute on Thursday, December 3rd. Of this Paper we are only able to reproduce some short abstracts. It was very fully illustrated, and there were a number of important appendices.

Modes of Failure of Materials.—That failure of materials cannot occur by real compression is fairly evident when we consider the fact that even a liquid such as water is practically incompressible. If we apply force from every direction, we do not cause rupture, as the material is not able to escape laterally. A number of experiments were made some years ago at McGill University by Prof. Frank D. Adams, and described in the *Proceedings* of the Royal Society, in which marble was placed in steel cylinders and compressed. It was found that this brittle material could be subjected to very great deformations without much reduction of its strength against thrusting stress, the material, indeed, showing a ductile character and ability to undergo plastic deformation when adequately supported in a lateral direction. The same thing has been found by experiments on concrete, as, for instance, some classic experiments by Prof. Ira H. Woolson. In one, concrete in a steel cylinder was bulged out under compression, yet when the encasing cylinder of steel was removed, the strength was found not to be appreciably different to that of concrete which had not been subjected to such deformation. It would seem that when complete lateral restraint is imposed, such as by enclosure in a cylinder, the cohesion is not developed, but, instead, the crystalline or directional materials undergo distortion by slipping on cleavage planes as a strictly ductile material would ordinarily.

The late Monsieur Armand Considère, in one of his papers on the resistance of hooped concrete, referred to experiments in which concrete had been placed to pass through the box of a hydraulic ram so as to be subjected to lateral pressure, and at the same time was subjected to vertical pressure. It was found that great crushing loads were thus enabled to be sustained without rupture. He also referred to an experiment in which that brittle material—glass—could be bent permanently without fracture when placed in a liquid under great hydraulic pressure. From this he drew the conclusion that the efficiency of hooping in increasing the crushing resistance to concrete was in the nature of a lateral support such as might be given by uniform lateral hydraulic pressure. The writer, however, thinks it would be more legitimate to regard the action of hooping in a different way.

He would prefer to consider the action to be as follows: The hooping either (1) applies a lateral compression equivalent to hydraulic pressure proportional to its extension under the lateral swelling defined by Poisson's ratio; (2) after initial cracking on the plane of rupture resists lateral motion of the parts by sliding on the plane of

rupture until failure occurs by the hooping breaking; or else (3) is of sufficient strength not to be able to be broken in this way, in which case the rupture of the concrete is confined to that extending between two adjoining hoops, that is to say, the concrete fails between the hoops like an extremely dumpy test specimen bedded on concrete. It is possible to calculate the effectiveness of hooping in these three respects.

It is desired to point out that the author has had insufficient opportunity to carry this mode of analysis of cohesive granular substance very far. As to how it would apply to ductile materials, and as to whether it would work out to give much the same result as Prof. Rejtö's theory has not yet been investigated, nor its relation to the resistance of pieces subjected to bending. It is hoped to present further contributions to the subject at a subsequent date.

Reinforced Concrete Beams.—Reinforced concrete beams are quite a distinct type in themselves, because they pass through two stages in their resistance to cross-breaking. In the first stage the steel remains uniformly connected to the concrete in which it is embedded. In the second stage the steel slips throughout the concrete for part of its length and the concrete cracks in the tension portion, so that the beam becomes converted into a form of construction which, in different modes of reinforcing, would appear to consist of either a concealed arch with a steel tie, a trussed beam analogous to the trussed timber beam, or a frame. Frames subjected to transverse loading are, of course, subjected to shearing forces, but the effect of the shearing forces is naturally very different on frames from that on a girder with a solid web or upon arches. A shearing force diagram only consists of the plotted values of the tendency to shear it asunder which the load exerts upon the structural member. We can, therefore, have a shearing diagram equally applicable to a beam, an arch, or a frame, but the manner of using the diagram for determining the internal stresses which result from the shear forces must be different in each case.

Reinforced concrete beams are quite a special form of construction, because the stresses are induced in them in various ways according to different modes of arranging the reinforcement, and according to the amount of reinforcement; that is to say, a reinforced concrete beam may prefer to act as a flat arch with a tie rod if it is able to hold up in such form with less stress than it would in some other fashion. On the other hand, if it acts as a beam, the conditions are different in the case where the stress is such as not to have ruptured the concrete by tension therein from the case where the concrete is cracked. Generally, some parts of all reinforced concrete beams are in the uncracked condition, and really require analysis on the lines of a homogeneous section, the materials, however, having different moduli of elasticity both for the concrete in tension and compression and the steel. In the majority of cases the reinforcement is insufficient in amount to prevent cracks occurring at some point or points in beams, and then when cracks appear it means that the steel has moved in the concrete, and that the construction either consists of a trussed beam or a framed beam in part or in whole. Where the reinforcement is inclined at the ends, there is an obvious analogy to a trussed timber beam. Such a type of reinforcement is quite appropriate for point loads. When the cranked bars are in combination with straight, horizontal bars, a practical type of reinforcement is provided which forms a sort of halfway-house, and can equally well resist point loads or distributed loads. In the latter case not only do the straight bars produce one sort of beam action, but the cranked rods resist the shear in that the inclined pull in the bars affords a vertical component at every part of the length to resist the shear.

Cracks that develop in reinforced concrete beams owing to the excessive stretching of the concrete generally follow the lines of the principal stresses, and point to the conclusion that they are the outcome of the diagonal tensile stresses that become developed in a homogeneous structure. The very presence of such cracks renders illogical the analysis of the distribution of the shear stress in reinforced concrete beams, which, I believe, was originally put forward in Germany, and appears in Prof. Emil Mörsch's book on "Der Eisenbetonbau," and also summarised in an appendix contributed by Mr. William Dunn to the British Joint Committee's Second Report on Reinforced Concrete; this analysis depends upon the section being homogeneous, and is based on the principle that the shear on vertical planes is accompanied by an equivalent

shear on horizontal planes. The shear is found by that analysis to be of uniform intensity below the neutral axis of the beam, because no normal stress is assumed to act upon the concrete below the neutral axis, and omitting this factor in the equations for shear in homogeneous beams gives the result stated. Seeing, however, that if there should be cracks in the beam extending practically to the neutral axis, obviously there can be no such horizontal shear stress across the cracks, and if there is no horizontal shear stress therefore vertical shear stress cannot exist there either. Indeed, the resistance to the shearing force of a beam must, in the presence of cracks, be altered to the manner in which an arch, a truss, or a frame resists shearing force. Diagonal cracks in nowise preclude the concrete from taking diagonal compression parallel to such cracks.

It was early found in practical work that it was advantageous to reinforce beams against shearing force in other manners than by trussing members. Either alone or in combination with such inclined reinforcements, web members (either vertical or diagonal) were provided.

Diagonal Tension and Compression in Beams.—The frame action of vertical and inclined web members generally belongs to two types. In the first case where vertical web members are employed the construction becomes a kind of N-truss, in which the concrete forms compression members and the steel the tension members. In the diagonal form the construction becomes practically a lattice girder. In both types of construction the frames may be, and often are, superimposed; that is to say, the spacing of the vertical members or of the diagonal web members is frequently fairly close. It is possible by attention to the design to put the web members very far apart and yet at the same time ensure frame action. In that case the concrete diagonals of the frame would become inclined at such a flat angle that the connection of the concrete to the tension member would not be very efficient unless special means were taken, while the steel would not be so economically arranged. If the diagonal compression in the concrete is at a sharp inclination exceeding the angle of friction, there is a tendency for the concrete to slide along the bar. This may be wholly or partly resisted by grip or adhesion of the concrete to the horizontal members. If, however, the inclination is very flat, this sliding tendency becomes great. In order to provide resistance to slipping, and for economy, the inclination and the stress of diagonals should be kept up, and, furthermore, precautions should be taken to prevent the web members sliding along the main tension member.

If, as has been said, the concrete diagonals are at a flat inclination, the friction induced by the inclined compression will be small, and the compression in the web concrete will come upon a small bearing area. For these reasons the author does not favour, in practice, the use of such very flat trussing, and advocates that web members be not spaced farther apart than the arm of the beam, so as to keep the diagonal compression inclined at 45 degrees, which is the most efficient angle for economising steel. The whole of the web concrete then becomes effective to take compression, and there is little or no tendency of slipping along the bars. Turned-up bars should have only slow bends, however, in order to give as much bearing as possible.

The author suggests that it is better not to adopt variable spacing of the web members, but to keep the same spacing throughout, for if we have variable spacing the system of superimposed frames, each equally sharing in carrying the load, will break down owing to the diagonal web members thereof not being all at equal angles, so that we should not be able to say how much was taken by each.

Another disadvantage in spacing web members far apart is that any point load applied between the panel points might cause crippling of the compression boom. The web members induce compression in the boom by means of their resolved horizontal components, which means that there is a shearing force at every horizontal section of the compression boom, but the presence of the compression component means that the condition is better than simple, pure punching shear.

Sometimes there is considerable torsion to be resisted by reinforced concrete. This torsion affords pure shear if there are no bending stresses induced at the same time.

It has been found from tests by Professor Mörsch, which have been confirmed by the author, that the punching shearing stress in reinforced concrete cannot be considered to be resisted to any appreciable extent by members crossing the plane.

Conclusion.—There are many other aspects in connection with the subject of shear. Though a good deal of ground has already been covered, there is any amount of room for continued investigation. It should be borne in mind that the theoretical analysis of the stresses in materials is in the nature of a speculation; it is an attempt, like all scientific theory, to explain the facts, but with the progress of investigation further facts come to light which cause us to modify the former theory. In that way there is a gradual improvement and greater exactitude about theories. But their chief purpose must always remain to draw the attention of the practical engineer to what is happening, likely to happen, or what may possibly happen, and in practical design the complication of refinements in theory are too great to permit of adoption. The practical engineer by study of the theory will, however, be able to make his own simplifying assumptions for approximate calculations, which will enable an economical and safe structure to be erected satisfactory in all the directions indicated by elaborate analysis.

THE INSTITUTION OF CIVIL ENGINEERS.

The following are Abstracts of Papers read at the ordinary meeting of the Institution on Tuesday, December 1st, 1914.

(I) TESTS OF REINFORCED CONCRETE STRUCTURES ON THE GREAT CENTRAL RAILWAY.

By JAMES BENJAMIN BALL, M.Inst.C.E.

THE Great Central Railway Company have completed within the last six years several considerable works in reinforced concrete, the more important of which include a bridge carrying a public road over their main lines at Ashton-under-Lyne—believed to be one of the heaviest reinforced-concrete bridges of its type yet constructed in England—a bridge carrying a new road and tramway over the Grimsby District Light Railway at Immingham dock, and several reinforced-concrete bridges in the dock-area at Immingham, as well as a large engine-shed at Immingham dock, the foundations and pits of which are constructed in reinforced concrete throughout.

REINFORCED CONCRETE OVERBRIDGE, ASHTON-UNDER-LYNE.

This bridge consists of parallel girders of three spans. The main girders also form the parapets of the bridge. On a level with the bottom booms are formed the main deck-beams or cross girders, and between these again are the smaller deck-beams parallel to the main girders, supporting the reinforced-concrete decking. The entire superstructure is reinforced with round bars on the Hennebique system.

The abutments at each end are of mass concrete, but the piers are braced and reinforced. The main girders only rest on the abutments and are not anchored down in any way; moreover, they are not continuous, there being a space of $1\frac{1}{2}$ in. between the girders over each of the intermediate supports for expansion: thus each span is independent, and consequently any slight settlement which may take place in the abutments or piers will not affect the stresses on the reinforcement, and in each case the full bending-moment effects due to both dead and live loads were taken into account.

Both tensile and compressive reinforcement was used in all the beams, the percentage of reinforcement in the main girders being exceptionally high, on account of the small area available in the compression flange.

The bridge was tested with a dead load of 1 cwt. per sq. ft. and a rolling load of two 16-ton traction engines, each drawing a lorry loaded with pig-iron to a weight of 32 tons, or a total moving load of 96 tons. In all cases the recovery was complete after the load was removed.

The working-stresses were limited to 700 lb. per sq. in. maximum compressive stress on the concrete, and 16,000 lb. per sq. in. tensile stress on the reinforcement.

The actual cost of the work as carried out in reinforced concrete amounted to £5,017. The estimated cost of a similar structure in steel and masonry was £8,390.

**REINFORCED CONCRETE BRIDGE CARRYING NEW ROAD AND TRAMWAY
OVER THE GRIMSBY DISTRICT LIGHT RAILWAY.**

This bridge, also designed on the Hennebique system, consists of a roadway 40 ft. wide, extending over two spans of 37 ft. 2 in. and 31 ft. 2 in. respectively. There are five main longitudinal beams, the two outer of which also carry a reinforced parapet. The parapets, however, are not designed to take any portion of the load-stresses.

Between these main beams cross-beams are arranged, which support reinforced decking.

The percentage of reinforcement ranges from 0.60 per cent. in the cross beams to 4.67 per cent. in the outer main beams of the longer span.

The bridge was tested with two moving tramcars, and no appreciable deflection was recorded on any of the beams.

It was designed to allow for the passage of two 40-ton boiler-trolleys drawn by a 15-ton traction-engine, that portion of the bridge not covered by the moving loads being loaded with 1 cwt. per sq. ft.

In computing the stresses, the various members were taken as being freely supported, and no allowance was made for the continuity of the beams or for the fixity of the ends, the allowable working-stresses being the same as in the case of the previous bridge.

The actual cost of this bridge carried out in reinforced concrete was £2,939. The estimated cost of a similar structure in steelwork and masonry was £3,500.

FOUR HIGH-LEVEL RAILWAY BRIDGES, IMMINGHAM DOCK.

These are all skew spans of the same construction, each bridge carrying four tracks over a siding for empty wagons, and forming practically a tunnel 86 ft. long.

The square span between the abutments is 15 ft. Both the wing-walls and abutments are reinforced, and there are in addition reinforced tie-beams at the bottom of the abutments, at the same spacing as the cross girders, embedded in a concrete raft.

The bridges were designed to carry axle-loads of 18 tons, spaced 6 ft. 6 in. apart, or 72 tons on the single-line span.

The average cost of these bridges in reinforced concrete was £3,024 each. The estimated cost of a similar structure in steelwork and masonry was £3,800 each.

**REINFORCED CONCRETE FOUNDATIONS AND PITS FOR ENGINE SHED
AT IMMINGHAM DOCK.**

This work consists of a reinforced-concrete raft, so spread that the load on the ground nowhere exceeds 10 cwt. per sq. ft., and the engine-pits form part of the raft.

The floor of the shed is carried on the pit walls; it consists of a reinforced-concrete deck between each of the pits, and is not in any way dependent on the earth filling underneath.

There are in addition fourteen independent reinforced-concrete engine-pits outside the shed, of a similar design to those in the shed, but without any flooring between.

Reinforced tie-beams are provided every 60 ft. apart between the pits.

Kahn bars were used throughout in the reinforcement of this work.

The actual cost of the work as carried out in reinforced concrete (engine-pits and foundations only) was £8,850. The amount of an actual tender received for brick pits on a reinforced-concrete slab was £12,920. An estimate for brick pits on concrete carried on timber piling was £15,150.

**(2) CONCRETING IN FREEZING WEATHER, AND
THE EFFECT OF FROST UPON CONCRETE.**

By JOHN HAMMERSLEY-HEENAN, Assoc.M.Inst.C.E.

The engineer who is called upon to carry out work in Canada during the winter finds that the methods of construction which were satisfactory in the summer will need considerable modification to suit winter conditions.

Concrete work, especially the lighter forms of reinforced concrete used in building construction, needs greater care and supervision. As a result of considerable experience gained during the last few years, it can be said that the freezing of concrete will not damage it if it has first had a chance to set under favourable conditions for about two days. The effect of the freezing is simply to delay the process of hardening, which will again proceed under suitable conditions, and will eventually attain its full strength. If concrete is frozen before it has commenced to set, it will not be injured if precautions are taken to prevent it from freezing again after it thaws until it is sufficiently hardened to withstand the effects of subsequent freezings. It is alternate freezing and thawing during the process of setting that causes the damage.

To meet the foregoing conditions when carrying out concrete work in winter it is necessary to devise means of mixing the concrete with materials freed of frost, placing it in the forms before it has commenced to freeze, and then protecting it and keeping it warm for about two days. After that it may be allowed to freeze without fear of its being damaged.

In the case of concrete-in-mass, of large bulk, it is unnecessary to apply external heat, as the large body of concrete will generate sufficient heat during the process of hardening to enable the mass to set; all that will be necessary is to protect the outside of the concrete so as to keep the heat in. This can best be done by covering the concrete with clean straw.

For light sections of concrete, such as in reinforced concrete, poured at a temperature not below 22° F., some engineers allow salt to be used in a proportion not exceeding 10 per cent. There are many arguments for and against its use. The author prefers not to use it, except in marine works when the concrete is mixed with sea-water, and the salt is admitted in that form. He has found that, instead of using salt, good results will be obtained for temperatures that do not fall below 22° F. by heating the water with a steam-hose taken from the mixer boiler, and when necessary placing a few coke or wood fires on the heaps of sand and crushed-stone, the usual precautions being taken to protect the concrete when in the forms, as described later.

For lower temperatures than those referred to above, greater precautions must be taken to heat the ingredients by means of steam coils or radiators.

The concrete having been mixed, and the portion of the work to be carried out decided upon, the floor immediately below it should be partitioned off with tarpaulins, and coke stoves arranged under the floor-slab, allowing about one stove to every 800 sq. ft. of floor-space. All loose dirt and snow must be removed from the forms with brooms, and a steam hose should be applied to remove all ice and frost, the steam playing continuously over the forms in advance of the concrete, thus warming them in readiness for the concrete. The concrete should be poured quickly and continuously, and as each section is completed a tarpaulin may be drawn over it, supported on wooden strips about 6 in. above the surface of the concrete. In most cases this protection will be sufficient, but during very cold weather it will sometimes be found necessary to form a sort of tent over the floor, in which extra stoves are placed to protect the workmen and the upper surfaces of the concrete.

Great care must be taken to have the fires kept burning continuously for two days, after which the concrete may be allowed to freeze without fear.

The work must be examined from time to time until it is found to be hardened sufficiently. During summer working, the author has allowed the supports from the underside of slabs to be removed in four days, but on other occasions four weeks have not been found to be too long.

There are many examples of concrete works which have stood the test of time without showing any signs of being affected by frost; but, on the other hand, a few cases have been reported of very serious corrosion due to the action of frost, such as bridge-piers and reinforced-concrete piles.

Judging from the information available at present, concrete exposed in air in a dry locality need not be affected by frost any more than good building stone, and probably it will stand much better. Concrete always submerged under water is protected and need cause no anxiety. But concrete alternately wetted and frozen must be protected from frost. On work which is being carried out at Halifax, Mr. John Kennedy, M.Inst.C.E., is protecting the concrete piles between high and low water with a covering of wood about 2 in. in thickness, which it is hoped will prevent the action of frost.

THE INSURANCE INSTITUTE OF MANCHESTER. REINFORCED CONCRETE CONSTRUCTION.

By S. BROADBENT.

A Prize for the best Essay on Reinforced Concrete Construction was recently awarded by the Manchester Insurance Institute to Mr. S. Broadbent. The Essay was read at a meeting of the Institute, and we are indebted to the Insurance Institute of Manchester for permission to reprint the following Abstract from the Paper.

In the earlier part of his essay the author deals briefly with the early history of reinforced concrete in England. Some remarks are made regarding compressive and shear stresses, the quality of the concrete, and the nature of the aggregate. He then goes on to deal with the question of steel, centering, reinforcement for beams and columns.

In the latter portion of his paper he deals with the fire protection of structural members. This portion of the paper we give as follows:—

THE FIRE PROTECTION OF STRUCTURAL MEMBERS.

After this brief description of the principles and practice of reinforced concrete construction, which in view of the space at my disposal must of necessity be a somewhat feeble and disjointed effort, a few remarks on the protection against fire of the main structural members will not be out of place.

It is a well-known fact that concrete, when subjected to intense heat of about 1,000 degrees Fahrenheit or over, for a continuous period will lose a part of the water taken up in crystallisation. This is known as dehydration, and when dehydration is complete the strength of the concrete is destroyed.

Dehydration, however, is a very slow process, and will, even in the most severe fire, only extend to the depth of one inch of the concrete, although cracks may extend further. Dehydrated material, being a poor conductor of heat and generally remaining in place for some time at least, it protects the mass of concrete from further dehydration.

It is not good practice, as a general proposition, to use the same material for structural duty and for resisting fire. The fire alone will give stresses enough for the material to resist without having to resist any structural load.

If the quantity of concrete on the outside of a reinforced column or beam is increased so as to allow a loss of the material during a fire without affecting the load-bearing quality of the structural member, the additional material added for fire resisting will participate in the load carrying, in spite of all that is done, and this would render it less able to resist a fire.

Furthermore, should the fire be so severe as to cause dehydration, cracks may extend beyond the dehydrated portion to the steel reinforcement, with obvious results, owing to the column or beam covering being an homogeneous material.

The ideal way to protect a structural member would be to construct that member as a unit of sufficient strength to withstand all structural loads, and then to place the fire protective covering on separately. This would allow of dehydration without affecting the load-bearing quality of the member, and, also, should cracks occur it is probable that they would not extend further than the covering owing to its separate construction.

For this reason, to ensure a first-class fire-resisting risk, a thickness of one inch of concrete on the outside of a structural member is, to my mind, sufficient for a separate fire protective covering for a reinforced member, while three inches should be regarded as necessary if the fireproofing is incorporated with the material for the column itself.

The above, however, should be regarded as a general statement, the covering necessary being dependent upon the class of trade for which the building is to be used and the likelihood or otherwise of a fire therein attaining a high temperature, and discretion should be used in specifying whenever an opportunity occurs, which I think will not be very frequent under existing conditions, owing to the extra cost involved by separately protecting a reinforced member.

It is my personal opinion that the rules applicable to buildings of fire-resisting construction could, with advantage, be revised and extended, as also could the discounts allowed, particularly in regard to buildings of reinforced concrete construction.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

THE PARKSVILLE CONCRETE DAM AND POWER PLANT.

THE accompanying illustrations show the Parksville concrete dam and power plant of the Eastern Tennessee Power Co. This installation, on the Ocoee River, at Parksville, Tennessee, with its 38,000 horse-power ultimate capacity and a distributing range of over 100 miles in every direction, takes its place among the great water-power developments of the country.

There are many interesting engineering features in its construction. Rising in the mountains of Northern Georgia, the Ocoee River flows through a region of heavy rainfall in a north-westerly direction to a point near Parksville, Tennessee, where it joins the Etowassee, in its course draining an area of more than 600 sq. miles.

At Parksville the mountains have been cut by Nature to form a deep and narrow gorge, above which the river flows for many miles through a broad and spacious basin—a location well suited to the purposes of water-power development. Where the valley is narrowest the great Ocoee Dam now stands spanning a gap of more than 800 ft. and filling the gorge to a height of 125 ft. from the river bed. The river, held captive in the valley above, forms a lake eight miles long, covering an area of $3\frac{1}{2}$ sq. miles in extent and containing 100,000 acre ft. of stored water.

In order to accomplish this, 155,472 cu. yds. of concrete were placed in the dam in exactly one year from the date of commencing. The dam is built throughout of solid cyclopean concrete, 840 ft. long at the crest, from 115 to 125 ft. thick at its base, with a spillway 362 ft. in length, capable of passing a flood of 45,000 cu. ft. each second.

It may be stated that the penstocks, through which that portion of the water utilised for power reaches the turbines, are tubelike cavities left in the masonry of the



THE PARKSVILLE CONCRETE DAM AND POWER PLANT.

dam. The water enters these at a point about 30 ft. below the crest, thus permitting only the top layer, about 20 ft. in thickness, to be used.

The main turbine chambers are likewise located wholly within the dam. The power house is situated immediately below the dam to the north of the spillway, its substructure being an integral part of the dam itself. The main building, 165 ft. long



THE PARKSVILLE CONCRETE DAM AND POWER PLANT.

and 35 ft. wide, contains the generating apparatus. Four main generating units and two exciter units are in operation, and the additional main unit for which provision was made in the original plans is now being installed.

The large turbines, which are mounted on horizontal shafts and connected direct with the generators, are designed to produce 5,400 horse-power each when operating



View of Finished Structure.
ALLENTOWN REINFORCED CONCRETE BRIDGE, U.S.A.

at the rate of 360 revolutions per minute under a pressure head of 98 ft. The generators have a capacity of 3,000 kilowatts and are operated at a voltage of 2,300. In the wing to the north of the main building are housed the transformers and high tension apparatus. The transformers, four in number, which are oil insulated and water cooled, increase the voltage from 2,300 to 66,000, at which the energy is carried over the transmission lines.

It is of interest to note that from quarries in the mountain side just below the dam stone was loaded by steam shovels to the dump cars, hauled by donkey engines to the crusher plant, and there reduced to the required sizes of both stone and sand. The materials thus applied by one process were then carried by belt conveyors to the storage bins over the concrete mixers, into which they were fed by gravity.

The cement was supplied to the mixers in a similar manner and the water from large storage tanks. From the mixers the concrete was loaded into buckets having a capacity of 2 cu. yds., hauled on flat cars to the dam, hoisted by derricks and dumped between the wooden forms to the place of its destination.

ALLENTOWN BRIDGE.

Our illustration shows one of the largest concrete viaducts recently erected in America. It is intended to carry the heavy inter-urban trains of the Lehigh Valley Transit Co., an inter-urban line running between Philadelphia and Allentown, and it is also to provide a highway toll crossing.

The bridge crosses a valley about 125 ft. deep at its deepest part.

Inclusive of approaches the bridge is 2,600 ft. long, the main part of which is made up of nine 120-ft. clear span arches and eighty 52½-ft. girder spans, arched for architectural effect. It carries a 32-ft. roadway with two 7-ft. sidewalks.

The chief engineer for this construction was Mr. W. W. Wysor, whilst the plans and specifications were prepared by Mr. B. H. Davis, of New York, who acted as consulting engineer during the work.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

The Engineering Society of China. Report of the Special Committee on Reinforced Concrete.

This society passed a resolution on December 20th, 1910, to appoint a special committee to enquire into the use of reinforced concrete, and this report is issued as the result of the investigations.

The tests that were conducted are not of universal interest, as all the materials employed were local and unobtainable in this country, and as the committee were hampered through lack of funds, the work done does not cover a very large field.

The provision of a suitable testing plant was a matter of great difficulty, and the purchase by the Municipal Council of Shanghai of a compression testing machine was helpful, as permission was obtained to use this. The Municipal Council also assisted financially by making a grant to the society, and without this timely help from the Council practically nothing could have been carried out.

The committee, in presenting the report, fully realise that a great deal still remains to be done, but it is hoped to continue the investigations at some future time.

Three brands of cement, all of which are Chinese, were tested, and these were found to comply with the British standard specification in each case. Six kinds of local sand were tested, together with six varieties of local stone, and, generally speaking, these were all found satisfactory, with the exception of the Soochow sands, which were shown to be inferior and not to be recommended.

The reinforcement tested consisted of ordinary round and deformed bars, and these tests were conducted, by the permis-

sion of the Chinese authorities, at the Shanghai Arsenal, the machine used being a single-lever machine (Wickstead's Patent) manufactured by Buckton and Co., Leeds, and capable of exerting a pull of 50 tons and testing material having a breaking strength of 300 tons per sq. in.

Concrete blocks were tested for compression strength, 610 specimens being dealt with in all, and after these beams and slabs were considered. These beams and slabs were reinforced in various ways and with different amounts of metal, and the concrete was also varied. The preliminary tests consisted of four beams, and all these failed by destroying the bond between steel and concrete. Final tests were then conducted with three sets of sixteen beams, half being made with gravel and half with broken-stone concrete. It was found that in no case did the concrete reach its maximum resistance, and the failure in several cases indicated the necessity of securing the bars at the ends.

In addition to the description of the tests with the reinforced slabs, notes are added on the fire-resisting qualities of reinforced concrete, the question of making a reinforced concrete structure impermeable to water, and the effect of electrolysis on metal embedded in cement concrete. Two appendices are given at the end of the report, the first of which is a specification for cement, and the second the certified analyses of the three different brands of cement that were used in the tests.

Although the work done by the committee refers exclusively to local materials, the report is nevertheless interesting, and will be of value to engineers engaged in reinforced concrete work in China.

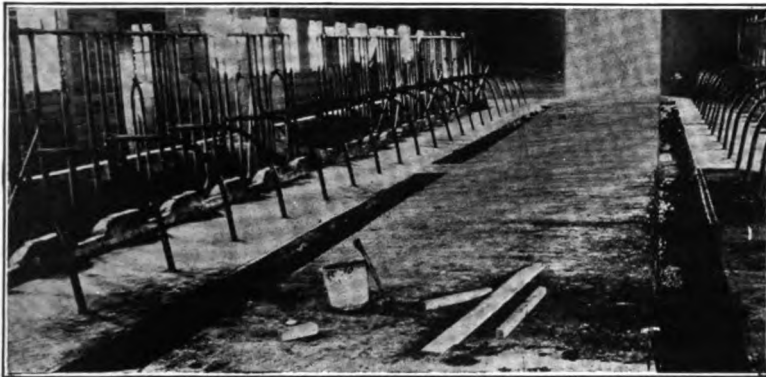
POPULAR USES.

Under this heading it is proposed from time to time to present particulars of the more popular uses to which concrete and reinforced concrete can be put, as, for instance, in the construction of houses, cottages and farm buildings. Previous articles will be found in our issues of December, 1912, and January, March, July, October and November of last year.—ED.

SANITARY FLOORS FOR THE DAIRY BARN.

Method of Laying Concrete Floors with Farm Labour.

It is of the utmost importance to the farmer that his barn floors should be sanitary and so made that they can be kept clean without much trouble. Concrete floors lend themselves admirably for this purpose. These floors are easily laid, and their cost is



SANITARY FLOOR WITH CONCRETE MANGER AND SWINGING STANCHIONS.

small. The plan here described is for a barn in which the two rows of cows stand heels towards each other, with a driveway between. It can be easily modified to the opposite arrangement. Likewise the method is adaptable to both old and new barns.

Planning and Grading the Floor.

For average conditions lay out the stalls on 3 ft. 6 in. centres and 4 ft. 6 in. in length from 6-in. manger wall to drop gutter. The manger is 2 ft. 6 in. wide at the top and 2 ft. at the bottom, with one face sloping up to the feed-alley floor. The depth is 7 in., measured from the stanchion setting, and 8 in. from the alley floor. The feed alley is 4 ft. 6 in. wide. The drop gutter has a width of 18 in.; it is 8 in. deep gauged from the stall floor, which is 2 in. higher than the 8 ft. driveway. For establishing grade lines a carpenter's spirit level (or a water level) and a chalk line are very helpful.

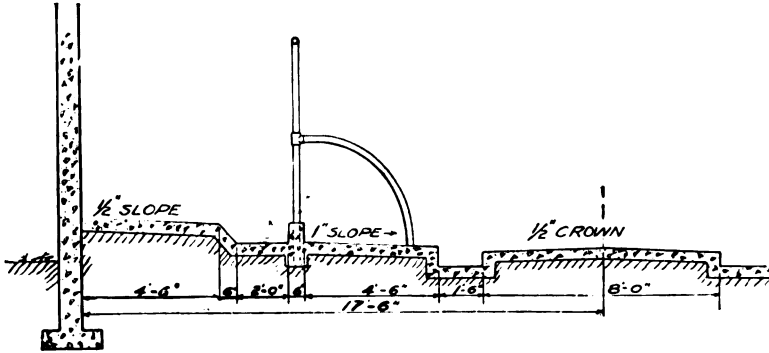
To prevent any possibility of the floor settling, remove all manure before grading the surface of the earthen floor. Carefully tamp back the dirt around water pipes and the drains which carry waste water and liquid manure to the water-tight concrete manure pit. Do all filling as long as possible before building the concrete floor. As a foundation for the stall floors proper, place a 6-in. thickness of coarse broken stone or screened gravel to keep the floor from direct contact with the ground. Since the stall floors are of prime importance, it is well to make them first. During this operation the unpaved driveway and alleys can be used as working space. Then finish, in the order named, the feed alleys, the driveways, the mangers, and lastly the gutters.

Mixing and Laying the Concrete.

For the plan given, 5 ft. 6 in. from the centre line of the driveway stake on edge (and to line and grade) a 2 by 12 in. plank, to serve as a form for the stall floor at the gutter. Likewise set a similar board, 5 ft. distant, to mould the 6 in. manger

wall and stanchion setting. It must be remembered that the stall floor has a slope of 1 in. towards the gutter and that the stanchion setting rises 7 in. above the stall floor. Drainage for gutters and mangers will be provided by sloping their concrete bottoms.

Proportion the concrete 1 bag of Portland cement to $2\frac{1}{2}$ cub. ft. of sand and 5 cub. ft. of crushed rock, or 1 bag of cement to 5 cub. ft. of clean pit gravel. At one operation



CROSS-SECTION OF CONCRETE DAIRY BARN FLOOR SHOWING USUAL DIMENSIONS.

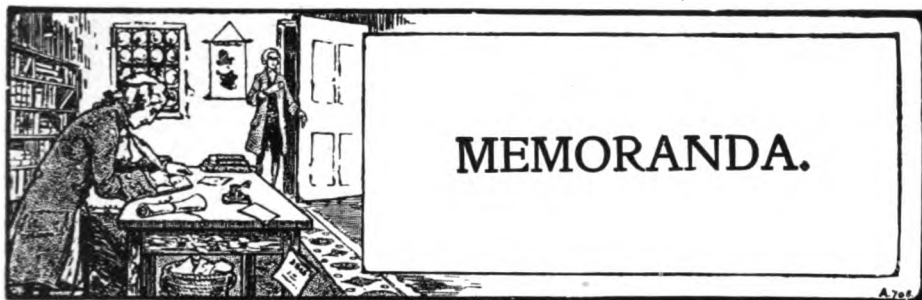
lay the full 5 in. thickness of the stall floor and finish three stalls the same as one section of sidewalk. No surfacing mortar is needed. A wooden float is best for finishing the floor. A steel trowel yields too smooth a surface and such a finish should always be roughened by brushing with a stable broom.

While the concrete of the three stalls is still soft, mould the stanchion setting (6 in. thick) upon it. As forms use the projecting 7-in. height of the 2 by 12-in. piece already in place and two 1 by 6-in. boards toe-nailed together so as to provide another 7-in. height and a bearing plate to rest on the green concrete. These forms may be made dish-shaped for swinging stanchions. Fill the forms with mushy wet concrete, trowel the surface, round the corners, and set the stanchion holders. Repeat the operation until all stall floors are completed. The feed alleys and driveway are easily built: they are merely rough-finished sidewalks. Place the waste-water outlets in the mangers at intervals of 28 ft. and give the bottom a slope of 1 in. towards each outlet for a distance of 14 ft. on each side of it. The drop gutters may be drained in like manner, or can be sloped slightly in one direction for their full length. For ease in cleaning, round all angles and corners (except at the bottom of the drop gutters) by applying a 1 to 2 cement-sand mortar immediately after removing the forms.

Caring for Cattle and Floor.

Regardless of the kind of floor, bedding of straw or litter is an absolute necessity: it keeps the cow clean and absorbs the valuable liquid manure. If the help cannot be depended on to bed the cows properly, it is advisable to use a removable wooden grating or platform. Cork bricks also give satisfactory results, but are somewhat expensive. They are set in a 2-in. depression in the floor and are held in position on all sides by the concrete acting as a curb.

With the proportions and thickness given above, 4 bags (1 barrel) of cement, 10 cub. ft. of sand (say $\frac{3}{4}$ cub. yard), and 20 cub. ft. of crushed rock (about $\frac{3}{4}$ cub. yard) will lay 45 to 50 sq. ft. of floor.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

American Concrete Institute.—The eleventh annual Convention of the Institute will be held at the Auditorium Hotel, Chicago, Ill., February 9-12th, 1915. This Convention will mark the completion of the tenth year of the existence of the Institute, and an especially interesting and profitable programme is being arranged. Some of the subjects to be discussed are: concrete roads, sidewalks, and bridges; concrete and reinforced concrete tests and design; concrete in art and architecture; plant management and costs.

The Institute has just issued its October-November Journal.

Part I. covers Institute notes, with announcement of the summary of the programme of the above-mentioned Convention.

Part II. covers the proceedings of the Institute, this issue containing:

Report of the test of a reinforced concrete flat slab floor by W. A. Slater, showing a comparison between actual and design stresses.

Report of the Committee on specifications and methods of tests for concrete materials, giving results of experiments conducted under the supervision of the Committee to determine the best form of concrete compression test piece and tests to obtain data to form a basis for standard specifications for sand, stone, etc.

Paper on some comparative corrosion tests of plastered metal lath, by J. C. Pearson, of the Bureau of Standards, giving details of tests and observations on the weathering effect on mortar panels of various proportions and ingredients.

Paper by Cloyd M. Chapman, giving data on lime putty and cream of lime, being the results of experiments to determine the necessary quantity of water to be added to hydrated or quicklime to give putty or cream of standard consistency. The data given is of great assistance to all engineers and contractors using plaster.

The Association of Consulting Engineers (Incorporated).—For some time past the Committee have had under consideration the advisability of issuing, for the guidance of the public and the profession, a scale setting out the usual professional fees, and the rules under which its members work. These have now been drawn up, and are available on application to the hon. secretary.

The British Engineers' Association.—The following is a short extract from speech by Mr. Wilfrid Stokes, chairman of the Executive Committee of the Association, on the necessity for reconstructing those Government Departments which deal with foreign trade, by establishing a new Board of Industry:—At the present moment attention naturally turns to our foreign trade conditions and how they are and may be affected by the War. There is a general feeling of uneasiness in connection with our present position with regard to foreign markets.

While it is universally acknowledged that the greatness of England depends upon her foreign trade, it is apparent that there is no adequate machinery provided by our Government Departments for properly extending and developing it.

It falls to the lot of the Foreign Office and the Board of Trade to deal with this matter, but both these departments are seriously hampered by the lack of funds and neither of them was ever constituted for this special purpose.

Just as we have an Army and Navy with their experienced technical advisers to defend our country, so we should have bodies of trained men all over the world to defend our trade under equally qualified leaders.

This trained force should be brought up with an intimate knowledge of our business requirements, and the leaders of it should be well paid so as to attract men of marked capacity and ambition after they have given proof of these qualities in business life.

A paid council composed of the best business men of the country and representatives of the High Commissioners of our self-governing dependencies, the India Office, and the Agents-General of the Colonies, should be formed and should meet at intervals to decide upon the general policy to be followed. It should be the duty of some of the members of Council to visit foreign countries and our Colonies to collect information at first hand. The head of this Council should be, *ipso facto*, a member of the Cabinet, and should be a permanent official chosen for his ability, and in receipt of a high salary. The Council should have a Parliamentary Secretary as its mouthpiece appointed by the Government in office, and the Department should be named the "Board of Industry."

Numerous qualified Trade Commissioners should be appointed in our dependencies and Colonies, and in foreign countries, with adequate staffs and ample office accommodation. The function of these Commissioners would be to promote the foreign trade of the Empire, thus relieving the Consuls of that portion of their work.

Our Ambassadors and Ministers should be provided with ample funds to further British trade interests by legitimate means.

The Commercial Attaché service should offer a separate and distinct career and should be made sufficiently attractive to secure the right men.

Owing to the fundamental character of these reforms, there is only one way of attaining them and that is by universal co-operation.

The cost of this war will, of necessity, be very great, so great indeed that its payment must be spread over a very long term of years. If our foreign trade is bad in the future this burden will be hard to bear. It is thus our obvious duty, as a community, to extend and improve our foreign trade to the utmost by all legitimate means.

By doing this we should hand on to posterity not only a heavy debt, but also a ready means of meeting the liabilities which have been forced upon us by the war.

An appeal is therefore made to all business men to do their best to further the reforms outlined above.

Lining Large Steel Coal Tank with Concrete.—As a preventive against the corrosive action of sulphur in coal and against abrasion, the Minnesota Steel Co. has lined a large circular steel coal tank at Duluth with concrete. The tank, which is approximately 55 ft. in diameter and 40 ft. high, with a capacity of 2,100 tons of coal, is used in supplying coal to the steel company's coke plant. For furnishing coal for the gas producers of the open-hearth furnaces, thirty-three parabolic bunkers, approximately 20 ft. wide, 12 ft. deep, and 17 ft. long, are provided. The coke-oven bins were lined with 5 in. of concrete and the producer bins with a 2-in. concrete lining. The cement gun was used for the purpose.

Iron lugs were provided at numerous places around the inside of the tank. To these vertical rods were securely wired in horizontal bands. The wire mesh was held away from the steel by a distance of $\frac{3}{4}$ in. Guniting was then applied. The first layer was shot through the mesh; it served to hold the wire cloth in place and covered and protected the steel plate. The second layer, bringing the coating to a total thickness of 5 in., was then applied.

The guniting applied consisted of a 1 : 3 mix of Portland cement and sand. It was shot through the hose by the gun with an air pressure of from 40 to 60 lb. Immediately before emerging from the nozzle the dry mixture was hydrated, the water coming through a separate and smaller hose—the quantity of water was under the control of the operator.—*The Engineering Record*.

Concrete for Purifier House at the Poole Gas Works.—In a recent issue *The Journal of Gas Lighting* gave a short account, with illustrations, of the new purifier house erected for the above Gas Co. The purifier house is of reinforced concrete on the "Hy-rib" system, and as the ground in the neighbourhood is very soft, the whole installation is "floated" on a reinforced concrete raft 2 ft. 6 in. thick, thus saving much expense in excavation. The total distributed load is 750 tons, and no movement has taken place since the raft has been loaded.

Cathedral in Reinforced Concrete at Georgetown, Demerara.—It has been decided to build the new Roman Catholic Cathedral at Georgetown in reinforced concrete on the Considère system. The construction is to be as light as possible; there will be a concrete raft over the whole area of the building so as to distribute the weight evenly over the whole ground covered.

Concrete Vaults.—Some of the American banking companies have recently adopted concrete for the construction of the bank vaults. The concrete is heavily reinforced and otherwise protected by electrical safety devices. The Depositors' and State Savings Bank in Chicago has a 2-storey vault, 20 ft. high and 18 ft. by 20 ft. in plan. The floor is a 12-in. slab of concrete reinforced with $\frac{3}{4}$ -in. rods placed on 6-in. centres in both directions, and near the centre of the slab additional protection is provided by steel rails laid at close intervals. Similar protection is provided in the 18-in. walls by the $\frac{3}{4}$ -in. reinforcing rods running in both directions. The roof is a 16-in. slab built like the floor. The Home Bank and Trust Co., also of Chicago, have also constructed a vault of reinforced concrete. It is 65 ft. long, 14 ft. wide, and 11 ft. high.

Concrete Lining for Steel Hull of a Steam Launch.—In the *Engineering News*, Mr. Horace M. Marshall, U.S. Engineer, Vicksburg, Miss., gives an interesting account of how the steel hull of a steam launch was preserved after it had been in use for thirteen years. In the first instance the hull was lined with cinder concrete reinforced with chicken wire. The hull was 65½ ft. long, 10½ ft. beam, and 4 ft. deep amidships; the plating was of soft steel 4½ lb. per sq. ft.; the draft was 20 in. to 24 in., according to load; and the wheel 30 in. in diameter. The wheel revolves in a tunnel, and when the boat is not under way only about two-thirds is below the water-line, but it is submerged when running; the water rises and fills the tunnel. When the boat was thirteen years old "pin-holes" developed all over the plating, and the plates were so rusted as to render the boat dangerous.

This state of affairs was met by a concrete lining. The hull inside was thoroughly scraped and cleaned. Chicken wire was spread over the ribs, and the cinder concrete rammed down through the mesh all over the bottom of the boat, extending well up on the sides above the water-line. The lining was from 4 in. to 5 in. thick at the keel and thinned towards the sides of the hull. The added weight was 8,000 lb., which increased the draught 3 in. to 6 in. The boat now worked satisfactorily for some three or four years, when the steel plates had grown so thin that it was feared the concrete would drop out; otherwise the vessel was in good condition.

This new difficulty was met by the application of a new steel skin or shell, which, however, necessitated the removal of most of the concrete, leaving only the concrete which was under the engine bed.

ENQUIRY.

A subscriber writes as follows:—"For a number of years I have been a subscriber to your magazine and would therefore ask you to let me know the best way of making an expansion joint in reinforced concrete. As a rule, I take it, expansion joints are not required, but it might be desirable to have such—for instance, in long trestle bridges, jetties, or reinforced concrete wharfs. If you published drawings and particulars in your magazine others might find it interesting, as most textbooks that I have come across have remarkably little to say on the subject in general.

REPLY.

Expansion Joints.—Expansion joints are only necessary when the area of concrete is large, or there is a large uninterrupted length such as a bridge or wall. The object of the joint is naturally to prevent cracks occurring, and it is really a means of

practically putting the crack in a selected position when executing the work, in preference to running the risk of possibly several cracks which would be unsightly. The exact method of forming the joint will vary according to the circumstances of each case, but generally it is formed by inserting a board, say, 1 in. wide, or less, as the case may be, and filling in the concrete on both sides. This board is withdrawn just as the concrete is setting and the space is filled in with sand or asphalt, or any material which will not be too rigid. Sometimes tarred felt or several thicknesses of tarred paper are put in to cut the concrete into sections, and these are allowed to remain. In any case, care must be taken that the space is not left in such a manner that water can penetrate to the reinforcement and cause rusting and decay. In a reinforced concrete over-bridge, Ashton-under-Lyne, where there are large parallel girders of three spans, the girders are cut over each intermediate support, and a space of $1\frac{1}{2}$ in. is left between the ends of the adjoining girders to allow for expansion. In the reinforced concrete retaining wall at the Royal Automobile Club, which is 228 ft. long, no allowance in the way of joints is made for expansion, but sufficient distribution rods are put in to take up the stress, and the cracks are then so distributed and fine as to be invisible to the naked eye.

A. L.



THE **VICTORIA** **CONCRETE MIXER**

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

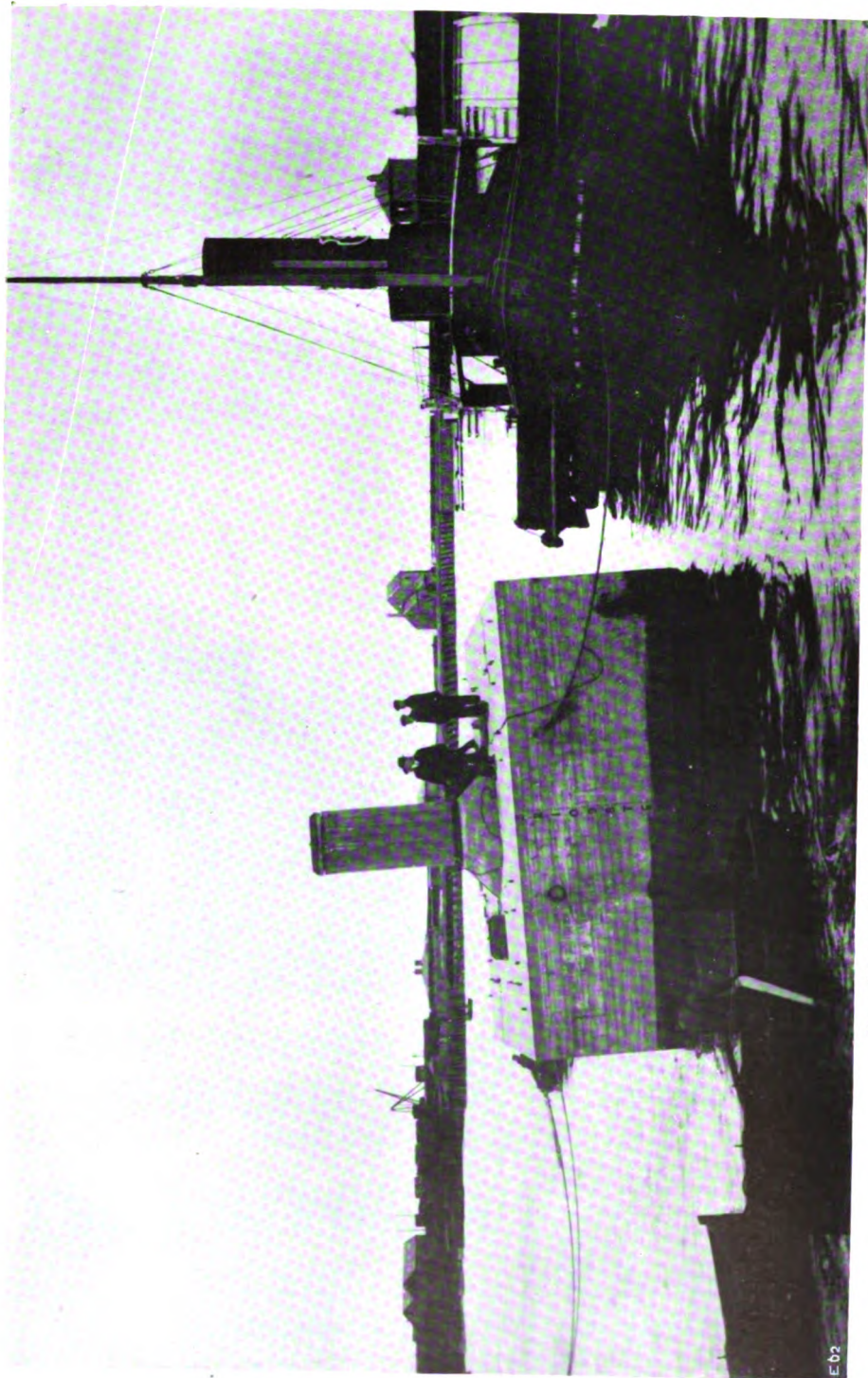
The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



View of Caisson after Launching and in Course of being Towed to the Site of the Works,
REINFORCED CONCRETE CAISSONS, SUNDERLAND DOCKS,
(For description see p. 63.)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X., No. 2.

LONDON, FEBRUARY, 1915.

EDITORIAL NOTES.

CONCRETE AND REINFORCED CONCRETE IN MUNICIPAL WORKS.

THE excellent paper read by Mr. H. J. Tingle, M.Inst.C.E., before the Concrete Institute, at their meeting last month, was particularly appropriate just at this time, when we are about to consider the re-planning and re-building of the cities and towns of Belgium, with their municipal works, which have been so ruthlessly destroyed by the invader.

Unfortunately, the title of Mr. Tingle's paper did not do justice to it. "The Application of Concrete in Modern Sanitation" was a title which scarcely gave one the impression that the author intended in this to deal with concrete and reinforced concrete sewers, reservoirs, sewage tanks, the manufacture of reinforced pipes, reinforced-concrete manholes, the cause of the failure of certain reservoirs, sewer outfalls, hydrolytic tanks in reinforced concrete, and many other important subjects. If the title had been "The Use of Reinforced-Concrete in Municipal Works," it would have been more appropriate.

As to the paper itself, Mr. Tingle certainly summarised in a most interesting manner a great deal of information, and one agrees with much that was included in the paper; but if one might offer a word of criticism, one would say the author attempted to include in one paper what could not be satisfactorily dealt with in half a dozen. Any of the subjects referred to would have been sufficient for one paper, and we trust he will be persuaded to develop the individual chapters. With regard to the use of concrete sewer tubes, undoubtedly these can be strongly recommended, but experience has shown that it is always desirable to haunch them up with concrete, as shown in *Fig. 1* of the paper; and they should never be laid in trench, as shown in the author's *Fig. 9*, without this concrete support, otherwise failure is invited. The collapse of concrete tubes through the omission of this safeguard would be a poor recommendation for the use of these tubes in sewer construction. If this precaution is taken, concrete tubes are admirable for sewers.

As to the reservoir which Mr. Tingle informs us failed, we think the design was certainly faulty, and that it would have been remarkable if the structure had not failed.

It is unfortunate that in selecting typical examples of the use of concrete and reinforced concrete in municipal engineering works, the author did not give us better samples of works constructed in these materials, but as already

indicated, to cover the ground fully, a series of lectures would be necessary. Concrete and reinforced concrete are undoubtedly eminently suitable for most municipal works, and excellent examples of their use in all kinds of such work may be found in this country and in America. Many instances of their use in this direction were given recently in a public lecture at the University of London.

As to temperature cracks in reservoir walls, these will occur unless expansion joints are put in or additional reinforcement to prevent them.

In conclusion, we would say that this paper, which was one of the best presented at the Institute, produced a very interesting discussion, and the general opinion was that provided they are properly designed, and the work is carried out in a workmanlike manner, concrete and reinforced concrete can be strongly recommended for municipal works.

REINFORCED CONCRETE IN EARTHQUAKE DISTRICTS.

It is reported in the daily Press that in connection with the earthquake in Avezzano a house said to be built of reinforced concrete withstood the shock and remained undamaged, and it is stated that it is the only one in the town which was left complete. We reproduce the accompanying illustration by courtesy of the *Daily Mirror*. We hope to be able to obtain further particulars and to publish them in a future issue.



HOUSE, STATED TO BE OF REINFORCED CONCRETE, WHICH WITHSTOOD EARTHQUAKE SHOCK AT AVEZZANO.

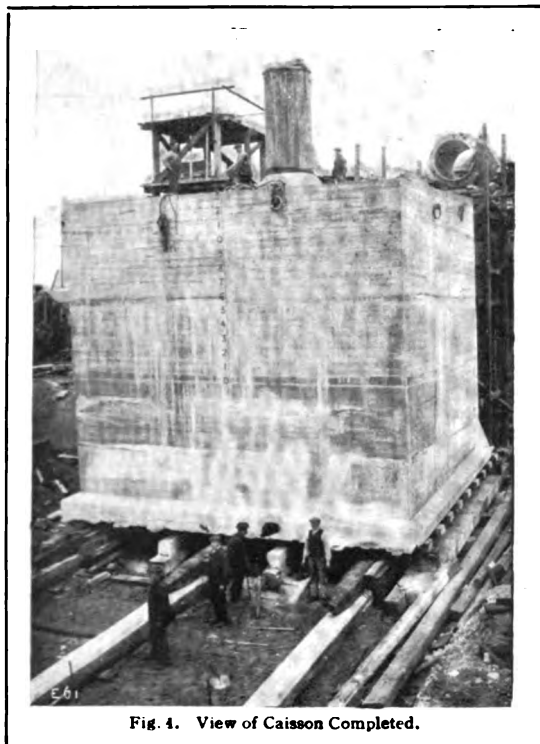


Fig. 4. View of Caisson Completed.

REINFORCED CONCRETE CAISSONS AT SUNDERLAND DOCKS.

We are indebted to Mr. Wm. Simpson, M.Inst.C.E., Engineer-in-Chief to the River Wear Commissioners, Sunderland, for the following interesting particulars and illustrations.—ED.

THE River Wear Commissioners, the Port of Sunderland Authority, have just constructed and launched at Sunderland two reinforced concrete caissons required for dock improvement works

which have many interesting and novel features.

Each caisson is of box form 26 ft. square, $23\frac{1}{2}$ ft. high, and is completely covered in on top (Figs. 1 and 5). A shaft, 3 ft. diameter, rises to a height of 11 ft. above the level of the roof, and is the means of access to the interior of the caisson. The bottom, walls, and roof of the caisson are stiffened with beams, and strutted with internal bracings. This is necessary, as the empty caisson has to sustain a maximum head of 36 ft. of water when sunk in position.

The caisson is required to carry a masonry superstructure, which will be built in the dry while the caisson is afloat. This top load will eventually sink the caisson in place, and it has been so arranged that when the caisson has grounded on the prepared base the top of the superstructure will stand clear above the ordinary water level of 34 ft., measured from the dock bottom. The caisson will then be filled up solid with concrete, passed through the shaft in the dry, and the remainder of the superstructure completed. In this way the whole of the work will have been built and hearted in the dry, thus ensuring good results in the quality of the work.

Grout pipes are provided, leading from the bottom to the top of the

caisson, so that a solid bearing can be formed between the prepared base and the bottom of the caisson.

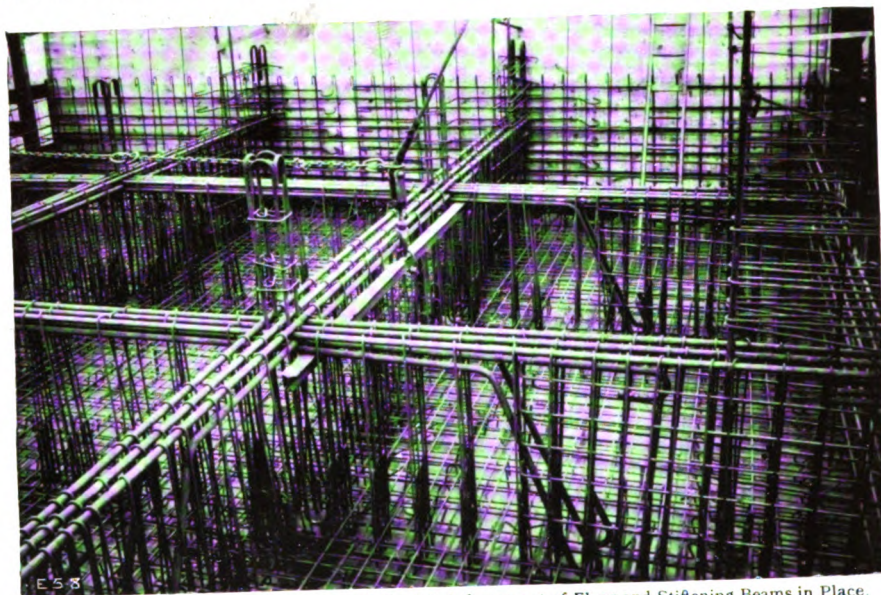


Fig. 2. View of Bottom of Caisson with Steel Reinforcement of Floor and Stiffening Beams in Place.

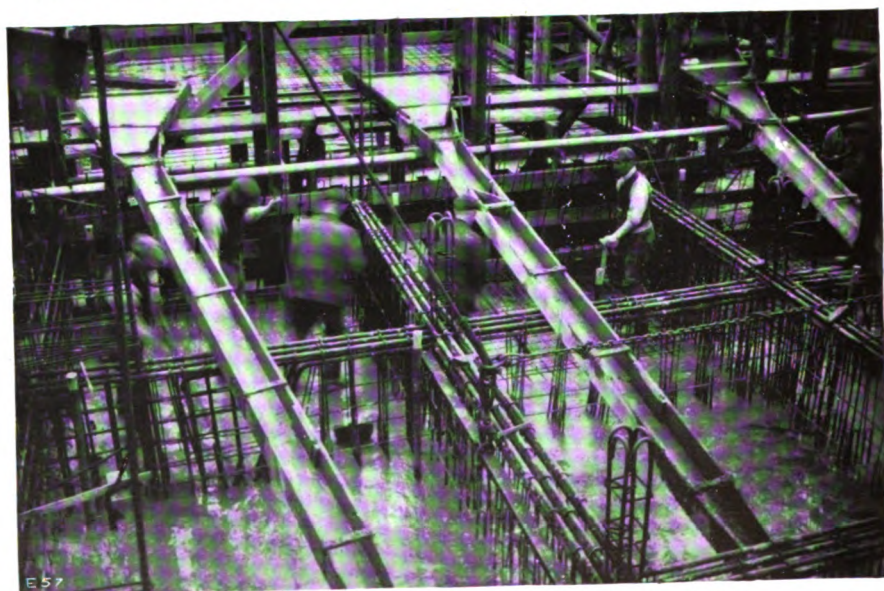
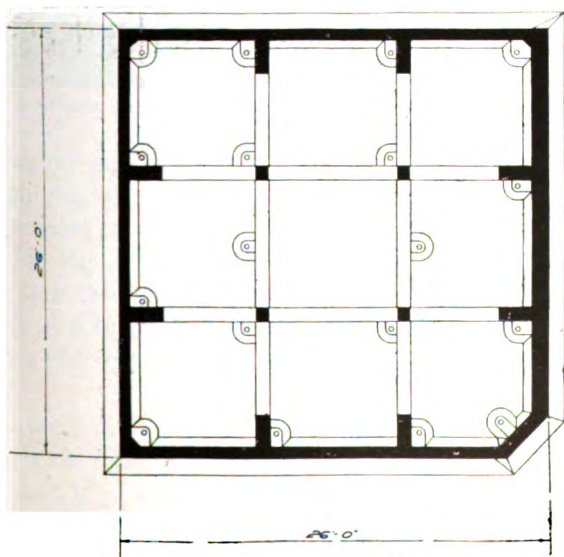
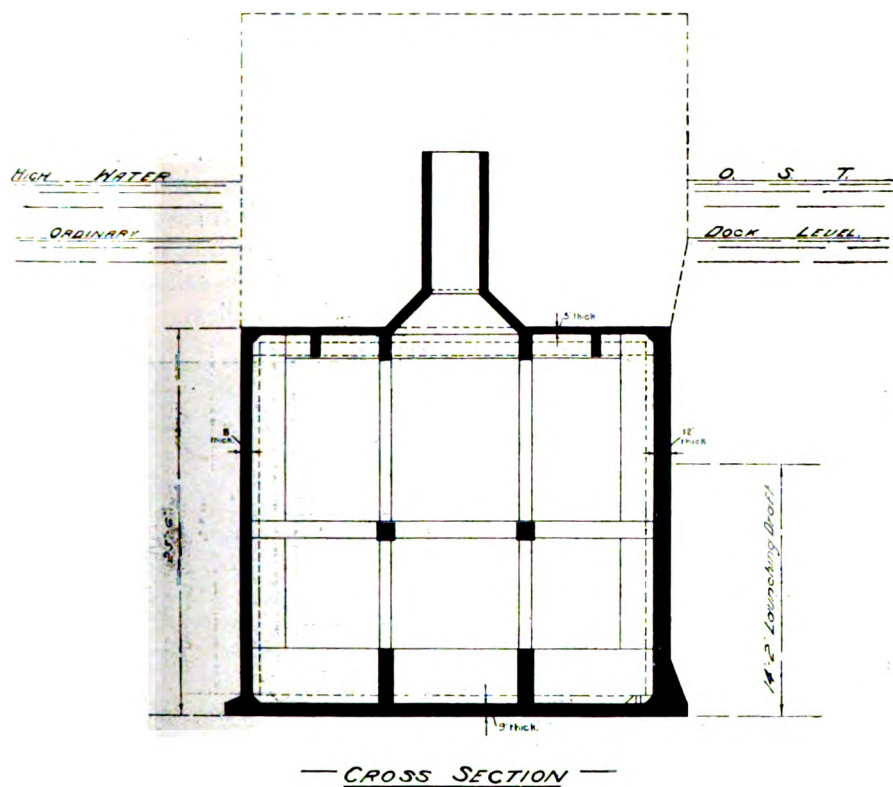


Fig. 3. View of Bottom of Caisson with Concrete Floor being put in Place.
REINFORCED CONCRETE CAISSONS, SUNDERLAND DOCKS.

The caissons have been built, one behind the other, on a flat foreshore of the River Wear, and were launched from ways, having a commencing



REINFORCED CONCRETE CAISSONS, SUNDERLAND DOCKS.

gradient of 1 in 15 and terminating at 1 in 12. The launching ways are three in number, and as the launching draft of each caisson is $14\frac{1}{2}$ ft., these ways had to be carried a distance of over 200 ft. from the site before the requisite depth of water was obtained. The caissons were first supported on building ways, placed between the launching ways, and just before launching the weight was transferred from the building to the launching ways by placing sliding cradles under each caisson, and blocking and wedging them up to the caisson bottom. The blocks and wedges of the building ways were then knocked out, and the caisson rested its entire launching weight of 300 tons on the sliding cradles.

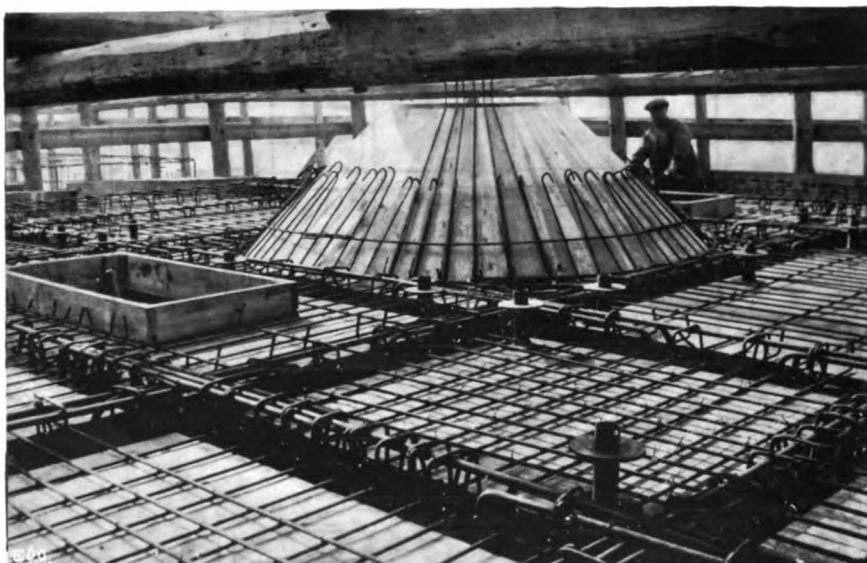


Fig. 5. View of Top of Caisson with Steel Reinforcement of Roof and Strengthening Beams being put in Place.
REINFORCED CONCRETE CAISSONS, SUNDERLAND DOCKS.

The caissons were constructed of concrete in the proportions of $2\frac{2}{3}$ cubic ft. granite chips to pass $\frac{3}{4}$ -in. mesh, $2\frac{2}{3}$ cubic ft. granite chips to pass $\frac{1}{2}$ -in. mesh, $2\frac{2}{3}$ cubic ft. sand, and 2 cubic ft. cement, making 10 cubic ft., or 4 to 1. This concrete was found to be 142 lb. per cubic ft. by actual weight after being mixed and dried. The concrete was placed in position between timber shutters, 2 in. thick, and about 6 ft. in height. These were shored in position off scaffolding which completely surrounded the caissons.

The concrete was mixed on the site by a "Smith" mixer, and distributed by means of timber spouts to the various points. As the heights of the walls increased, the mixed concrete was hoisted by a crane to platforms placed at suitable heights. The timber moulds and shutters were used alternately for each caisson, so that their construction proceeded simultaneously.

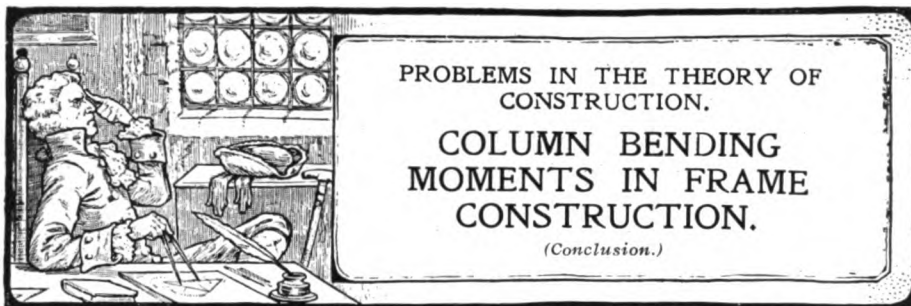
When launched, the caissons floated practically at the calculated draft, and were quite watertight.

The caissons have been built on the Considère system by the River Wear Commissioners' own workmen, to the designs and under the supervision of Mr. Wm. Simpson, M.Inst.C.E., Engineer-in-Chief to the River Wear Commissioners, Sunderland.

The accompanying photographs show the caissons under construction and in course of launching and being towed to the site of the works in the docks (*frontispiece*).



Fig. 6. View of Caisson in Course of Launching from the Ways.
REINFORCED CONCRETE CAISSONS, SUNDERLAND DOCKS.



By EWART S. ANDREWS, B.Sc.Eng.(Lond.).

This article is continued from page 16 in our last issue.—ED.

WE now come to the consideration of a continuous beam of two equal spans, each carrying a uniformly distributed load but of different intensities, the beam being rigidly connected to one or more of the columns.

Case I. Neglecting the Stiffness of the Columns.—If we neglect the effect of the stiffness of the columns upon the slope of the beam, we can investigate the bending moments and slopes as follows:—

Referring to Fig. 7, in which w_1 and w_2 are the intensities of the loads on the spans 1, 2; 2, 3 respectively, 1a2, 2b3 are the "free bending-moment diagrams," i.e., those which would occur if the beam were not continuous, and 1c3 is the reverse bending-moment diagram caused by the continuity, the resultant diagrams being the difference between the two as shown shaded in the figure.

Treating the B.M. diagrams as imaginary loads, as before, we find the slopes by calculating the imaginary reactions.

The area of the parabola 1a2

$$= P_1 = \frac{2}{3} \cdot \frac{w_1 l^2}{8} \times l = \frac{w_1 l^3}{12}.$$

The area of the parabola 2b3

$$= P_2 = \frac{2}{3} \cdot \frac{w_2 l^2}{8} \times l = \frac{w_2 l^3}{12}.$$

The area of the $\Delta 1c2 = \Delta 3c2 = P_3 = B_2 \frac{l}{2}$; this acts through the centroid of the Δ , i.e., at a distance $\frac{l}{3}$ from 2.

Then if r_1, r_2, r_3 are the imaginary reactions at 1, 2, 3 we have by taking moments about 3, for the right-hand span, and calling clockwise moments positive,

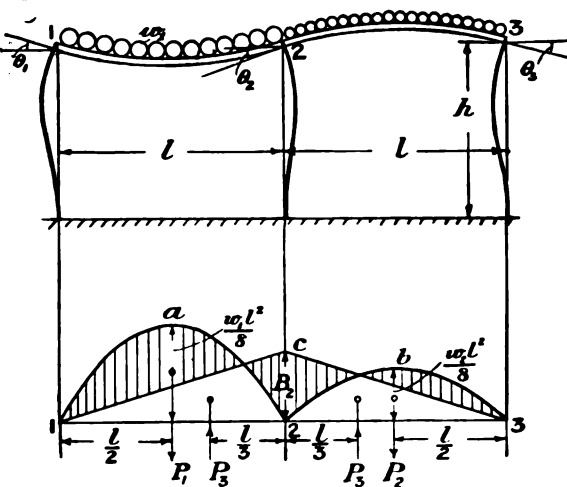


FIG. 7.

$$\begin{aligned}
 r_1 \cdot l &= P_1 \cdot \frac{2l}{3} - P_2 \cdot \frac{l}{2} \\
 &= \frac{B_1 \cdot l^2}{3} - \frac{w_1 l^4}{24} \\
 \text{i.e., } r_1 &= \frac{B_1 l}{3} - \frac{w_1 l^3}{24} \quad (1)
 \end{aligned}$$

Similarly for the span 1, 2 we have by taking moments about 1

$$\begin{aligned}
 r_2 &= \frac{w_1 l^3}{24} - \frac{B_1 l}{3} \quad (2) \\
 \therefore \frac{2B_1 l}{3} &= \frac{w_1 l^3}{24} + \frac{w_2 l^3}{24} \\
 \text{i.e., } B_1 &= \frac{(w_1 + w_2) l^2}{16} \quad (3)
 \end{aligned}$$

putting this result in either of equations (2) or (3) we get

$$r_2 = \frac{(w_1 - w_2) l^3}{48} \quad (4)$$

Knowing the value of B_1 we might have reasoned in this way: $r_1 = \frac{1}{2}$ area of parabola $1a2 - \frac{2}{3}$ area of $\Delta 1c2$.

$$\begin{aligned}
 &= \frac{P_1}{2} - \frac{2}{3} P_2 \\
 &= \frac{w_1 l^3}{24} - \frac{2}{3} \left(\frac{w_1 + w_2}{16} \right) \frac{l^3}{2} \\
 &= \frac{w_1 l^3}{24} - \frac{(w_1 + w_2) l^3}{48} \\
 &= \frac{(w_1 - w_2) l^3}{48}
 \end{aligned}$$

$$\begin{aligned}
 \text{Similarly } r_1 &= \frac{P_1}{2} - \frac{P_2}{3} \\
 &= \frac{w_1 l^3}{24} - \frac{(w_1 + w_2) l^3}{96} \\
 &= \frac{(3w_1 - w_2) l^3}{96} \quad (5)
 \end{aligned}$$

$$\begin{aligned}
 \text{and } r_2 &= \frac{P_3}{3} - \frac{P_2}{2} \\
 &= \frac{(w_1 + w_2) l^3}{96} - \frac{w_2 l^3}{24} \\
 &= \frac{(w_1 - 3w_2) l^3}{96} \quad (6)
 \end{aligned}$$

If therefore I_B is the moment of inertia of the beam, we have

$$\theta_1 = \frac{(3w_1 - w_2) l^3}{96EI_B} \quad (7)$$

$$\theta_2 = \frac{(w_1 - w_2) l^3}{48EI_B} \quad (8)$$

$$\theta_3 = \frac{(w_1 - 3w_2) l^3}{96EI_B} \quad (9)$$

But if B_D is the bending moment at the top of the column we have previously shown that

$$B_D = \frac{kFI_c \theta}{h}$$

$$\therefore B_{D1} = \frac{k(3w_1 - w_2)l^2 \cdot II_c E}{96E \cdot I_B h}$$

$$= \frac{k(3w_1 - w_2)l^2}{96s_1} \quad (10)$$

$$B_{D2} = \frac{k(w_1 - w_2)l^2}{48s_2} \quad (11)$$

$$B_{D3} = \frac{-k(3w_2 - w_1)l^2}{96s_3} \quad (12)$$

As B_{D1} will always come less than B_{D3} , assuming that w_1 is greater than w_2 , we will not consider it further, and we regard equations (10) and (11) as giving the column bending moments for the exterior and interior columns respectively.

In the common case in which the columns are fixed at the ends, $k=4$ giving the results:—

$$\text{B.M. at top of exterior column} = \frac{(3w_1 - w_2)l^2}{24s_1} \quad (13)$$

$$\text{B.M. „ „ interior „} = \frac{(w_1 - w_2)l^2}{12s_2} \quad (14)$$

Reaction on Columns.—Since the reaction at the end of a beam is given by the slope of the B.M. curve and the effect of continuity is to decrease the slope of the B.M. curve at the end 1 by an amount $\frac{B_2 - B_1}{l}$, the reaction at 1 for the ordinary case of non-continuity will be diminished by an amount equal to $\frac{B_2}{l}$ because $B_1 = 0$.

$$\therefore R_1 = \frac{w_1 l}{2} - \frac{B_2}{l}$$

$$= \frac{w_1 l}{2} - \frac{(w_1 + w_2)l^2}{16l} \quad \left[\text{from (3)} \right]$$

$$= \frac{7w_1 l}{16} - \frac{w_2 l}{16} \quad (15)$$

$$\text{Similarly } R_3 = \frac{7w_2 l}{16} - \frac{w_1 l}{16} \quad (16)$$

And since $R_1 + R_2 + R_3$ must be equal to the total load $(w_1 + w_2)l$, we have

$$R_2 = \frac{5(w_1 + w_2)l}{8} \quad (17)$$

$$\text{We may also write } R_2 = \frac{w_1 l}{2} + \frac{w_2 l}{2} + \frac{2B_1}{l}$$

Numerical Example.—We are now in a position to illustrate our results as far as we have gone up to the present. Take the case illustrated in Fig. 8, the main beams being 15 ft. apart. The dead load may be taken as 80 lbs. per sq. ft. and the super load as 200 lbs. per sq. ft.

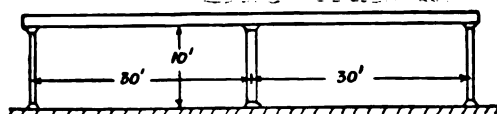


FIG. 8.

Then for the left span loaded and the right unloaded we have $w_1 = 280 \times 15 = 4,200$, $w_2 = 80 \times 15 = 1,200$ lbs. per ft. run. The maximum bending moment occur when both

spans are loaded and is equal to $\frac{w_1 l^2}{8} = \frac{4,200 \times 30 \times 30 \times 12}{8} = 5,670,000$ in.-lbs. and taking a safe stress of 16,000 lbs. per sq. in. we have :—

$$\text{Section modulus required} = \frac{5,670,000}{16,000} = 354.4 \text{ in.}^3$$

A compound girder composed of two 14-in. $\times$ 6-in. $\times$ 46-lb. I beams and plates to make 14 in. $\times$ 1½ in. in each flange will give a modulus of 353.3 and a moment of inertia of 3,020 in.⁴ which will do well.

Now consider the centre columns; when both spans are fully loaded, the load on the central column = $\frac{5}{8} \times 8,400 \times 30 = 157,500$ lbs. = 70.3 tons. A 14-in. $\times$ 6-in. $\times$ 57-lb. I beam ($I_c = 532.9$) will just carry this by the L.C.C. formula for fixed ends, and in the usual method of designing without taking account of the bending stresses in the column, this section would be regarded as amply strong. It will be best to arrange this with its long axis parallel to the beam, so as to carry the bending stresses better.

The outside columns will only carry $r_1 \times 70.3 = 21.9$ tons. A 6-in. $\times$ 5-in. $\times$ 25-lb. I beam [$I_c = 43.61$] will carry this.

Now take the case of the left-hand span only covered with the live load. Then the load on the centre column = $\frac{5}{8}(4,200 + 1,200)30 = 45.2$ tons.

By equation (14) the bending moment at the top of the interior column = $\frac{(w_1 - w_2)l^2}{12s_2}$

$$\text{In our case } s_2 = \frac{I_B \cdot h}{I_C \cdot l} = \frac{3,020 \times 10}{532.9 \times 30} = 1.9 \text{ approx.}$$

$$\therefore B_D = \frac{3,000 \times 30 \times 30 \times 12}{12 \times 1.9} = 1,420,000 \text{ in.-lbs. approx.}$$

$$\therefore \text{Bending stress} = \frac{B_D}{\frac{532.9}{7}} = \frac{1,420,000}{532.9} \times 7 = 18,700 \text{ lbs. per sq. in.}$$

This is more than the safe stress apart from the direct stress as a column and, unless our direct method of calculations is far from the truth, the column is not sufficiently strong. If the bending moment on the exterior column be calculated by equation (13), that column will be found to be more highly stressed still.

Case II. Allowing for Stiffness of Central Column but neglecting that of Exterior Columns.—In this case we allow for the stiffness of the central column but neglect that of the exterior columns; for the purpose of finding the bending moment on the central column this is equivalent to assuming that the connection of the beam to the

columns 1 and 3 does not alter the slope of the beam at the ends compared with the slopes which would occur if the ends merely rested upon supports.

Fig. 9 shows the resultant B.M. diagram for the beam. There is a sudden change in the reverse B.M. diagram due to the bending moment B_{D2} taken by the central column. Con-

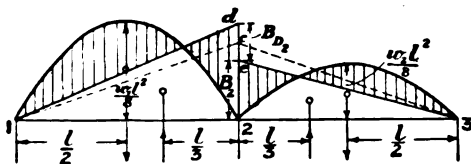


FIG. 9.

sidering the span 2, 3, we have by moments about 3 for finding the imaginary reactions, as before,

$$r_3 l = \frac{B_2 l^2}{2.3} - \frac{w_2 l^4}{24} \quad (18)$$

and considering the span 1, 2 and taking moments about 1 we get

$$r_1 l = \frac{w_1 l^4}{24} - \left(\frac{B_2 + B_{D2}}{2} \right) \frac{l^2 l}{3} \quad (19)$$

$$\text{i.e., } \frac{2B_2 + B_{D2}}{3} = \frac{(w_1 + w_2) l^2}{24}$$

$$\text{or } B_2 = \frac{(w_1 + w_2) l^2}{16} - \frac{B_{D2}}{2} \quad (20)$$

$$\begin{aligned} \therefore \frac{r_2}{l} &= \frac{EI_B \theta}{l} = \frac{(w_1 + w_2) l^2}{48} - \frac{B_{D2}}{6} - \frac{w_2 l^2}{24} \\ &= \frac{(w_1 - w_2) l^2}{48} - \frac{B_{D2}}{6} \end{aligned} \quad (21)$$

$$\text{but } B_D = \frac{kEI_{c2} \theta}{h} \quad \therefore \frac{r_2}{l} = \frac{EI_B \theta}{l} = \frac{hI_B}{lI_{c2}} \cdot \frac{B_{D2}}{k} = \frac{s_1 B_{D2}}{k}$$

$$\therefore \frac{s_1 B_{D2}}{k} = \frac{(w_1 - w_2) l^2}{48} - \frac{B_{D2}}{6}$$

$$B_{D2} \left(\frac{s_1}{k} + \frac{1}{6} \right) = \frac{(w_1 - w_2) l^2}{48}$$

$$B_{D2} = \frac{(w_1 - w_2) l^2}{48 \left(\frac{s_1}{k} + \frac{1}{6} \right)}$$

$$= \frac{k(w_1 - w_2) l^2}{8(6s_1 + k)} \quad (22)$$

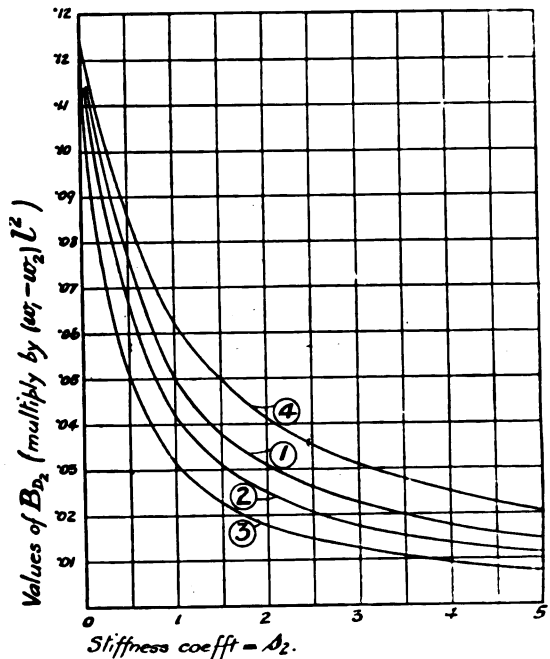
Taking the values of k previously given for the standard cases 1 to 4, we get the values which are plotted in Fig. 10.

To find the total reverse B.M. $B_{D2} + B_2$, on the beam, we have from equation (20)

Total reverse B.M. =

$$\begin{aligned} B_{D2} + B_2 &= B_{D2} + \frac{(w_1 + w_2) l^2}{16} - \frac{B_{D2}}{2} \\ &= \frac{B_{D2}}{2} + \frac{(w_1 + w_2) l^2}{16} \end{aligned} \quad (23)$$

To draw the B.M. diagram, therefore, we set up 2c, Fig. 9, equal to $\frac{(w_1 + w_2) l^2}{16}$ (the mean height of the two parabolas), this being the same as when the stiffness of the centre column is neglected; we then look up from Fig. 10 the value of B_{D2} for the given method of fixing the ends and set down and up ce, cd equal to $\frac{B_{D2}}{2}$ and complete the diagram as shown.



1. Lower end of column fixed.
2. " " " hinged.
3. " " " slopes equally in opposite direction.
4. " " " " " same " "

FIG. 10.

To find the B.M. on exterior columns, say 1, we have by moments about 2,

$$\begin{aligned} r_1 l &= \frac{w_1 l^4}{24} - \frac{(B_2 + B_{D_2})}{2} \cdot \frac{l}{3} \\ \frac{r_1}{l} &= \frac{s_1 B_{D_1}}{k} = \frac{w_1 l^3}{24} - \frac{B_2 + B_{D_2}}{6l} \\ &= \frac{w_1 l^3}{24} - \frac{B_2 + \frac{1}{2} B_{D_2}}{6l} - \frac{B_{D_2}}{12l} \\ &= \frac{w_1 l^3}{24} - \frac{(w_1 + w_2) l^3}{6l} - \frac{B_{D_2}}{12l} \\ \therefore \frac{s_1 B_{D_1}}{k} &= \frac{(3w_1 - w_2) l^3}{96} - \frac{B_{D_2}}{12l} \end{aligned} \quad (24)$$

From the values of B_{D_2} given in Fig. 10 B_{D_1} can be found similarly,

$$\frac{s_3 B_{D_3}}{k} = \frac{(w_1 - 3w_2) l^3}{96} + \frac{B_{D_2}}{12l} \quad (25)$$

Reaction on columns.—The reaction on the centre column is given by

$$\begin{aligned} R_2 &= \left(\frac{w_1}{2} + \frac{w_2}{2} \right) l + \frac{B_2 + B_{D_2}}{l} + \frac{B_2}{l} \\ &= \left(\frac{w_1 + w_2}{2} \right) l + \frac{2}{l} \left(B_2 + \frac{B_{D_2}}{2} \right) \\ &= \frac{w_1 + w_2}{2} l + \frac{2}{l} \left(\frac{w_1 + w_2}{16} \right) l, \quad \left[\text{from (20)} \right] \\ &= 5 \left(\frac{w_1 + w_2}{8} \right) l \end{aligned}$$

The reaction on the centre column will therefore be the same as before.

On the other columns we have

$$\begin{aligned} R_1 &= \frac{w_1 l}{2} - \frac{B_{D_2} + B_2}{l} \\ &= \frac{w_1 l}{2} - \frac{(B_{D_2} + B_2)}{l} - \frac{B_{D_2}}{2l} \\ &= \frac{w_1 l}{2} - \frac{(w_1 + w_2) l}{16} - \frac{B_{D_2}}{2l} \quad \left[\text{from (20)} \right] \\ &= \frac{(7w_1 - w_2) l}{16} - \frac{B_{D_2}}{2l} \end{aligned}$$

This is the previous value minus $\frac{B_{D_2}}{2l}$, and R_3 will come equal to the previous value

plus $\frac{B_{D_2}}{2l}$; the effect therefore of the stiffness of the interior column is to take a slight amount of load off the more heavily loaded exterior column and to transfer it to the other side.

Numerical example.—Taking the same numerical example as before we have in this case

$$\begin{aligned} B_D &= \frac{k(w_1 - w_2) l^3}{8(6s + k)} = \frac{4(w_1 - w_2) l^3}{8(11.4 + 4)} \\ &= \frac{3,000 \times 30 \times 30 \times 12}{2 \times 15.4} = 1,050,000 \text{ in.-lbs. approx.} \end{aligned}$$

$$\therefore \text{Bending stress} = \frac{1,050,000 \times 7}{532.9} = 13,800 \text{ lbs. per sq. in.}$$

This is somewhat less than before, but still very serious; this justifies our statement that the assumptions involved in the case (1) will err on the side of safety.

*Case III. Allowing for stiffness in all the columns, but taking the two exterior columns as of the same stiffness and method of fixing. This case leads us into more troublesome expressions, but can be investigated as follows:—*Fig. 11 illustrates the resulting B.M. diagram.

Take first the first span and consider the reverse B.M. diagrams as divided into two triangles, we have:—

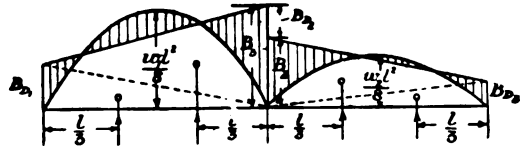


FIG. 11.

Taking moments about 1,

$$r_2 l = \frac{w_1 l^4}{24} - \frac{B_3 l^2}{3} - \frac{B_{D1} l^2}{6} \quad (26)$$

$$r_1 l = \frac{w_1 l^4}{24} - \frac{B_3 l^2}{6} - \frac{B_{D1} l^2}{3} \quad (27)$$

Dividing (27) by 2 and subtracting from (26) and dividing through by l^2 we get after simplifying

$$B_3 = \frac{w_1 l^2}{12} - \frac{4r_2}{l} + \frac{2r_1}{l} \quad (28)$$

Putting this in (26) and simplifying we have

$$B_{D1} = \frac{w_1 l^2}{12} + \frac{2r_2}{l} - \frac{4r_1}{l} \quad (29)$$

Next considering the second span and reversing signs as before we have

$$r_2 l = -\frac{w_2 l^4}{24} + \frac{B_2 l^2}{3} + \frac{B_{D1} l^2}{6} \quad (30)$$

$$r_3 l = -\frac{w_2 l^4}{24} + \frac{B_2 l^2}{6} + \frac{B_{D1} l^2}{3} \quad (31)$$

Treating these in the same way as (26) and (27) we get

$$B_2 = \frac{w_2 l^2}{12} + \frac{4r_2}{l} - \frac{2r_3}{l} \quad (32)$$

$$B_{D1} = \frac{w_2 l^2}{12} - \frac{2r_2}{l} + \frac{4r_3}{l} \quad (33)$$

We next have the three relations

$$B_{D1} = \frac{k r_1}{s_1 l} \quad (34)$$

$$B_{D2} = B_3 - B_2 = \frac{k r_2}{s_2 l} \quad (35)$$

$$B_{D3} = \frac{-k r_3}{s_3 l} = \frac{-k r_3}{s_1 l} \text{ (because } s_1 = s_3 \text{)} \quad (36)$$

$$\therefore \text{ From (29) and (34) } \frac{k r_1}{s_1 l} = \frac{w_1 l^2}{12} + \frac{2r_2}{l} - \frac{4r_1}{l}$$

$$\text{i.e., } \frac{r_1}{l} \left(\frac{k}{s_1} + 4 \right) = \frac{w_1 l^2}{12} - \frac{2r_2}{l} \quad (37)$$

Similarly from (33) and (36)

$$\frac{r_3}{l} \left(\frac{k}{s_1} + 4 \right) = -\frac{w_2 l^2}{12} - \frac{2r_2}{l} \quad (38)$$

Putting (28) and (32) in (35) we have

$$\frac{k r_2}{s_2 l} = \frac{(w_1 - w_2) l^2}{12} - \frac{8r_2}{l} + \frac{2r_1}{l} + \frac{2r_3}{l} \quad (39)$$

Putting results (37) and (38) in this we have

$$\frac{k r_2}{s_1 l} = \frac{(w_1 - w_2) l^2}{12} - \frac{8 r_2}{l} + \frac{2 w_1 l^2}{12 \left(\frac{k}{s_1} + 4 \right)} + \frac{4 r_2}{l \left(\frac{k}{s_1} + 4 \right)} - \frac{2 w_2 l^2}{12 \left(\frac{k}{s_1} + 4 \right)} + \frac{4 r_2}{l \left(\frac{k}{s_1} + 4 \right)}$$

$$\text{i.e., } \frac{r_2}{l} \left\{ \frac{k}{s_2} + 8 - \frac{8}{\left(\frac{k}{s_1} + 4 \right)} \right\} = \frac{(w_1 - w_2) l^2}{12} \left\{ 1 + \frac{2}{\frac{k}{s_1} + 4} \right\}$$

Simplifying and multiplying through by $\left(\frac{k}{s_1} + 4 \right)$

$$\frac{r_2}{l} \left\{ \left(\frac{k}{s_2} + 8 \right) \left(\frac{k}{s_1} + 4 \right) - 8 \right\} = \frac{(w_1 - w_2) l^2}{12} \left(\frac{k}{s_1} + 4 + 2 \right)$$

$$\frac{r_2}{l} = \frac{(w_1 - w_2) l^2}{12} \left\{ \frac{\left(6 + \frac{k}{s_1} \right)}{\left(\left(\frac{k}{s_2} + 8 \right) \left(\frac{k}{s_1} + 4 \right) - 8 \right)} \right\}$$

$$= \frac{(w_1 - w_2) l^2}{12} \cdot \frac{(6s_1 + k)s_2}{\{ (8s_2 + k)(4s_1 + k) - 8s_1s_2 \}} \quad (40)$$

∴ from (35)

$$B_{D_2} = \frac{k r_2}{s_2 l} = \frac{(w_1 - w_2) l^2}{12} \left\{ \frac{k(6s_1 + k)}{8s_1 + k(4s_1 + k) - 8s_1s_2} \right\} \quad (41)$$

We will also find an expression for B_{D_1} .

From equation (37)

$$\frac{r_1}{l} \left(\frac{k}{s_1} + 4 \right) = \frac{w_1 l^2}{12} + \frac{2 r_2}{l}$$

$$\text{and } \frac{r_1}{l} = \frac{s_1 B_{D_1}}{k}$$

$$\therefore B_{D_1} \left(1 + \frac{4s_1}{k} \right) = \frac{w_1 l^2}{12} + \frac{2 r_2}{l} \quad (42)$$

$\frac{r_2}{l}$ having been found by equation (40) the value of B_{D_1} can thus be found readily and B_{D_3} can be found similarly from equation (39).

Numerical Example.—We can now find the value of the B.M. on the centre column for the numerical example that we have already considered. We have $s_2 = 1.9$

as before and $s_1 = \frac{3,020 + 10}{43.61 \times 30} = 23$ nearly, also $k = 4$

$$\therefore B_{D_2} = \frac{(w_1 - w_2) l^2}{12} \left\{ \frac{4(138 + 4)}{(15.2 + 4)(92 + 4) - 350} \right\}$$

$$= \frac{3,000 \times 30 \times 30 \times 12}{12} \times .38$$

$$= 1,030,000 \text{ in.-lbs. nearly.}$$

This is not appreciably different from the previous case in which we neglected the stiffness of the exterior columns. Suppose that the exterior columns were more stiff, say that their moment of inertia were half that of the interior column, we should then have $s_1 = 3.8$ and

$$B_{D_1} = \frac{(w_1 - w_2) l^2}{12} \left\{ \frac{4(22.8 + 4)}{(15.2 + 4)(15.2 + 4) - 57.8} \right\}$$

$$= \frac{3,000 \times 30 \times 30 \times 12}{12} \times .347$$

$$= 930,000 \text{ in.-lbs. nearly.}$$

In the extreme case in which the exterior columns are infinitely stiff we have $s_1 = 0$ and equation (41) gives $B_{D_2} = \frac{(w_1 - w_2)l^2}{12(8s_2 + k)}$

This corresponds to the case in which the extreme ends of the beam are fixed or "built-in."

In the case in which the column extends above the beam as well as below we may replace $\frac{k}{s}$ in the various equations by $\frac{k_a}{s_a} + \frac{k_b}{s_b}$ where the suffixes a, b indicate the quantities for the columns above and below the beam respectively.

The above treatment will, we think, demonstrate that there is considerable algebraic complexity involved in a rigorous treatment for the various conditions that may arise in practice, but that some such treatment is very necessary if high bending stresses in the columns are to be adequately met.

If designers are convinced of the importance of the problem it should not be a matter of very great difficulty to devise rules that are simple to apply and sufficiently accurate to ensure adequate resistance against the bending stresses.



A REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.

An interesting example of Reinforced Concrete construction is the new building for the Ford Motor Co., Chicago. We are indebted to our contemporary, "Engineering News," U.S.A., for the description and illustrations here given.—ED.

THE new assembling and service building for the automobile business of the Ford Motor Co., at Chicago, presents some unusual and interesting features in its design. The building has a 164-ft. front on 39th Street and extends south on Wabash Ave. 232 ft., with a provision for a further extension to a length of 460 ft. It is six stories high, without basement. The building has reinforced-concrete columns, floors and roof. Except on the alley side, no outside concrete is exposed, the exterior being of brick, terra-cotta and glass. The proportion of window area is large, in accordance with modern factory construction, and makes up about 60 per cent. of the exterior.

The building rests on a deep bed of sand. As the property is surrounded on three sides by streets and on one side by an alley, there is little danger of the foundations being undermined by adjacent excavations. The foundations are all spread footings designed for a soil pressure of 3,500 lb. per sq. ft., including the percentage of the live-load required by the Chicago building code. The footings under the wall columns on the street sides form a continuous girder 5 ft. 6 in. deep and from 7 ft. 4 in. to 8 ft. wide, *Fig. 1*. The widened portion (section *B B*) is to carry two intermediate columns one story high. The top of the footing is 15 in. below the first floor.

The column foundations on the alley side and also at the south end of the building are simple spread footings. The typical exterior footings are 14 ft. 3 in. square. Between the column footings on the alley side a 12-in. curtain wall of concrete was built for a depth of 3 ft. 9 in. below and 3 ft. 6 in. above the first-floor line. The first floor is laid directly on the ground.

The footings at the south end of the building were designed for full panel loads in order to take care of the future loads that will come on these columns when the building is extended. The weight of the present south wall is now carried upon these footings, but when the building is extended this weight will be replaced by the weight of the floor construction extending to the next row of new columns. Between the footings at the south end of the building is a concrete curtain wall 13 in. wide and extending $4\frac{1}{2}$ ft. below the first-floor level. This carries the weight of the 13-in. brick wall of the first story. The interior column foundations are stepped footings reinforced by two layers of steel bars placed near the bottom. The size of the bases varies from 13 ft. 6 in. to 17 ft. 9 in. square.

The floors are of the Akme system of flat slab or beamless construction, and are 11 in. thick. They are designed for a live-load of 150 lb. per sq. ft., with panels 28 by 25 ft. At the south end, the floor slabs extend 4 ft. beyond the centre line of the columns. The temporary south wall is carried on cantilever extensions of the floors, which are extended to this distance to provide for carrying the next panel of floor slabs when the building is lengthened. A pair of steel angles forming a Z-section is secured to the face of each floor slab

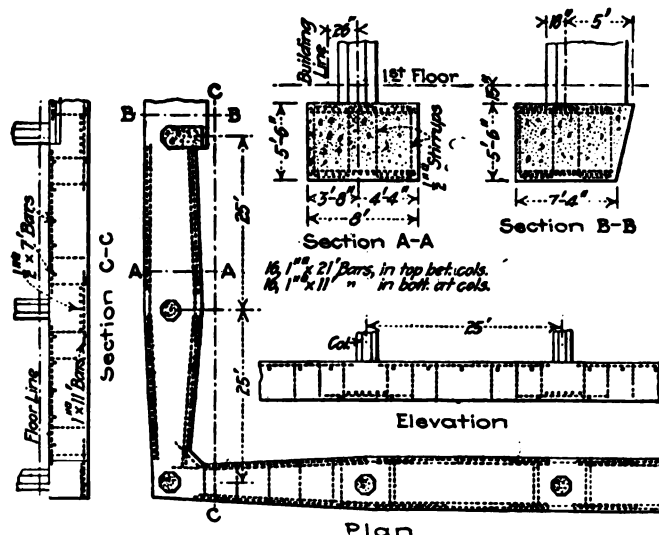


Fig. 1. Continuous Footing.

REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.

by anchor bolts, thus forming a shelf to support the end of the future slab, as shown in Fig. 2. The angles are at present incased in concrete which has been cast separate from the general floor slab so that when the extension is made this concrete can be easily removed, leaving the steel shelf angle projecting.

A mezzanine floor covers a portion of one panel

between the first and second floors. This is supported on two columns and suspended by four sets of hangers. Each hanger consists of two or three 1-in. round rods 11 ft. 8 in. long, with top and bottom bearing plates made of 5-in. channels. These plates are embedded in the concrete of the mezzanine floor and the floor above.

The columns are all of octagonal section, varying from 40 in. to 24 in. outside diameter. The column capitals are also octagonal, with a flare of 45 deg. from the vertical and a top diameter of 5 ft. 6 in. Above the flaring head is a column plate 7 ft. square by 9 in. thick, the lower edge of this plate being bevelled. The columns are spaced 28 ft. c. to c. longitudinally and 25 ft. laterally, except that in the central bay of the building, where the railway track is supported, the columns are spaced 35 ft. c. to c., laterally.

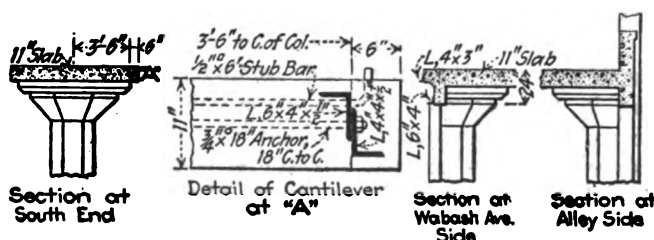


Fig. 2. Column Heads and Floor Slabs with Detail of Cantilever Extension at South End.

REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.

There are no spandrel girders required in this floor construction, but in

some instances the concrete section has been dropped down a few inches below the ceiling along the wall to carry the terra-cotta trim for the exterior. The east, or alley wall of the building is made of concrete instead of brick, and on this side the concrete columns, floors and spandrel walls were cast integrally.

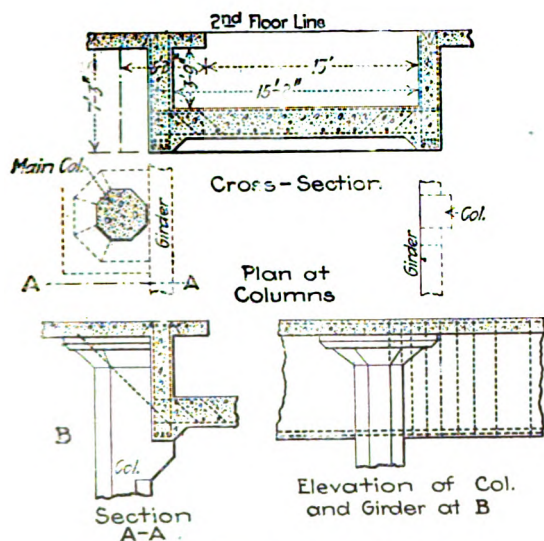


Fig. 3. Reinforced Concrete Bridge Carrying Track.
REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO.,
CHICAGO.

This track is in the central 35-ft. bay of the building, and is carried in a concrete trough 16 ft. wide and about 5 ft. deep. The main track girders are 7 ft. 3 in. deep and 16 in. thick. Between these girders is a bottom slab 21 in. thick, carrying the track. One of the main girders is carried on the main columns, while the parallel girder is carried on independent columns spaced 28 ft. c. to c.

The second floor projects a short distance beyond one of the main track girders, thus providing a protected space under the overhanging floor for wires and heating pipes.

The track is laid with wood ties and stone ballast. The whole bridge struc-

Concrete Track Girders.—

The only girders in the building are those carrying the railway track, the crane runways and the monitor roof. The railway lines adjacent to the building are elevated, and a switch track runs through the building at such an elevation as to bring the car floors level with the second floor of the building.

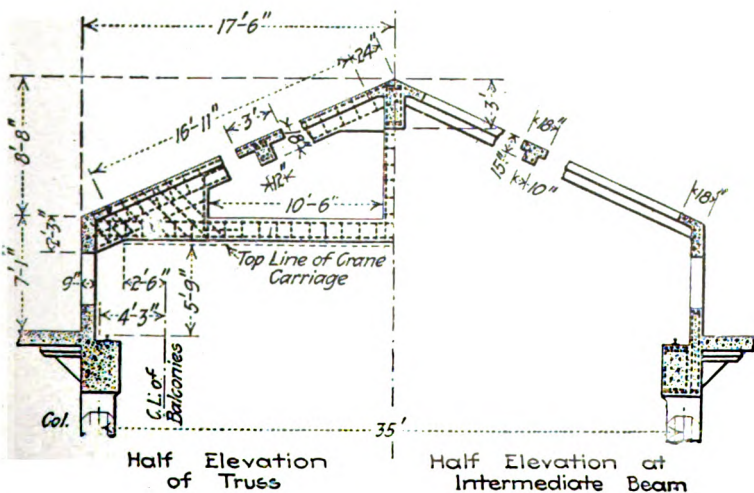


Fig. 4. Reinforced Concrete Trusses Carrying Monitor Roof.
REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.

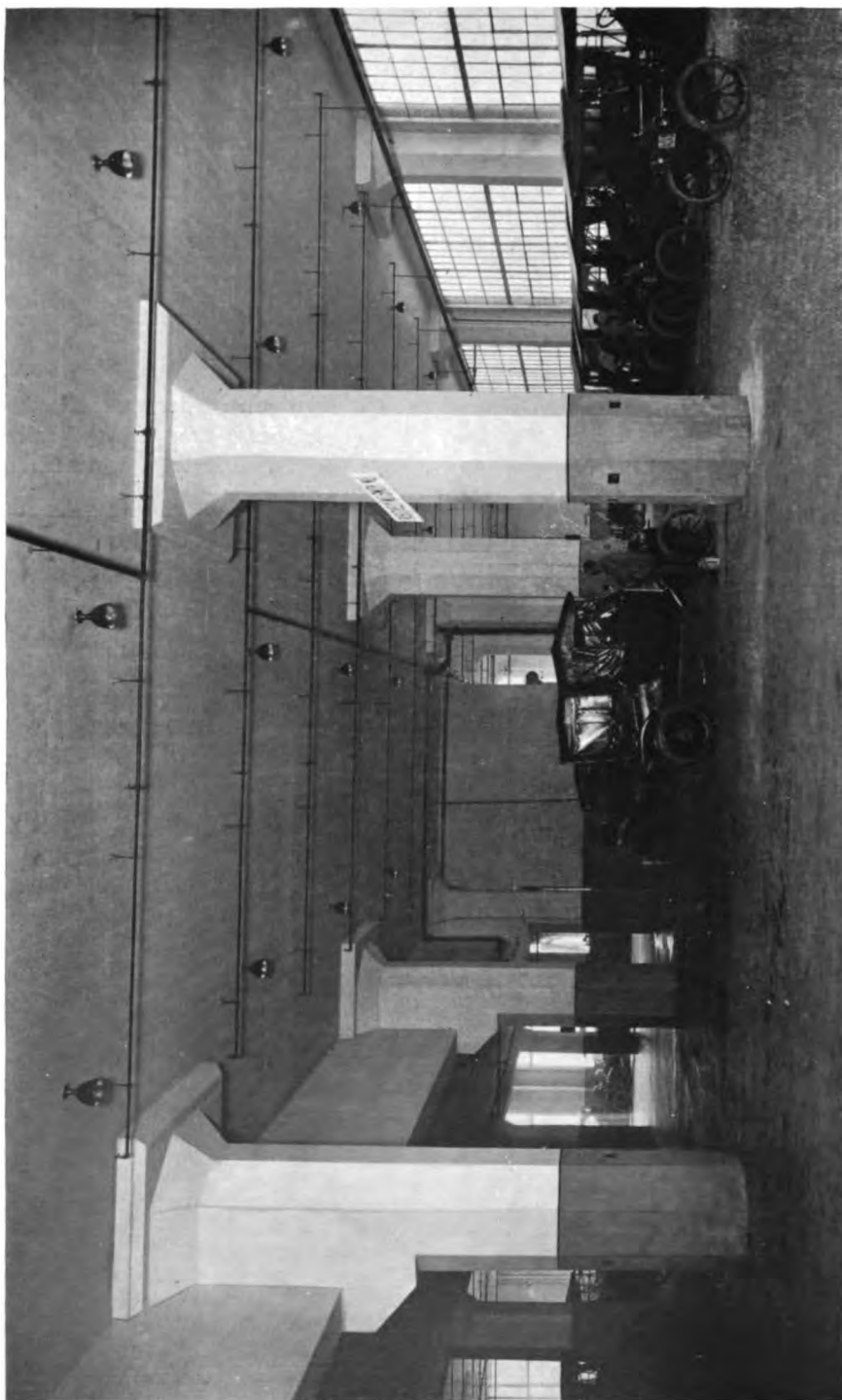


Fig. 5. View of First Floor.
REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.

ture design is in accordance with the recommended practice of the American Railway Engineering Association. The construction is shown in *Fig. 3*, and *Fig. 5* is a view of the first floor, showing at the left one of the heavy track girders supported on the main columns.

Concrete Balcony Floors and Roof Trusses.—In the central panel, 35 ft. wide, there is an open space from the second floor to the roof, except at the extreme north end where there is a floor panel 28 by 35 ft. at each floor. This open well or craneway is provided with an overhead bridge crane which runs the entire length of the building and serves all floors by means of cantilever balconies extending 8 ft. 6 in. from the column centres. These balconies are extensions of the floor slabs, and are of varying lengths; 14 ft. on the 6th floor, 38 ft. on the 5th floor, 42 ft. on the 4th floor, and 56 ft. on the 3rd floor. So there is always an open space 14 ft. long by 6 ft. 8 in. wide on each balcony from which to receive and deliver materials or automobiles from the bridge crane.

One of these 8½-ft. cantilever slabs 42 ft. long was tested with a uniform test load of 310 lb. per sq. ft., equal to 2,600 lb. per lin. ft., in addition to weight of construction. Under this load the deflection was $\frac{5}{32}$ in. at the outer edge, after the load had been on for 24 hours.

Below the roof level the two central rows of columns carry the reinforced concrete longitudinal girders for the runway tracks of the bridge crane. Above these runway girders rise the side walls of the monitor roof. The roof is carried by reinforced-concrete trusses, having a span of 36 ft. and spaced 28 ft. apart. As shown in *Fig. 4*, these trusses are of the king-post type with a rise of 8 ft. 6 in. The bottom chords of the trusses are simply tie rods incased in concrete, but the section was made sufficiently strong to act as a beam to carry its own weight. The bottom chord and post are rectangular, while the upper chords are of T-section, 18 in. deep and 36 in. wide over the flanges.

The trusses are supported at the ends by the walls of the monitor roof, and are connected at the ridge by longitudinal T-girders 36 in. deep. The roof between each pair of trusses is supported by two intermediate rafters of T-section, dividing the area into three panels 7 ft. 4 in. by 16 ft. long on each side. Each of these panels is covered by a copper and glass skylight. At the north end of the building an arched concrete wall or girder replaces the concrete truss. This is 9 ft. deep at its ends and 14 ft. deep at the centre, and is faced on the exterior with brick and terra-cotta.

Fig. 6 shows several of these special features of the building, being an interior view of the central craneway. Freight cars will be noticed standing on the depressed track, with car floors level with the second floor of the building. It will be seen that there is a wide unloading space on one side of the car and that the cantilever balconies on the opposite side overhang the cars slightly. This craneway or court will be inclosed entirely with steel sash and wire glass, with sliding doors opening from each floor on to the balconies.

Construction Plant.—During the construction of the building the concrete materials were brought in by cars on an elevated trestle and unloaded into storage bins south of the building site and beneath the track. From these bins the sand and gravel was delivered by means of a belt conveyor to elevated bins

above the measuring boxes, which in turn were above the concrete mixer, so that measured quantities of gravel and sand were discharged into the mixer,



Fig. 6. View Showing Interior of Building with Cantilever Balcony Floors and Concrete Roof Trusses.
REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.

together with the proper amount of cement and measured quantity of water for each batch. The 1-yd. mixer discharged the concrete into a 2-yd. hopper feeding a 1-yd. bucket in the elevator tower. The mixing plant and tower were

located at the south end of the building. All of the concrete was spouted directly to the floor forms, except that the concrete for the monitor roof was placed by means of carts.

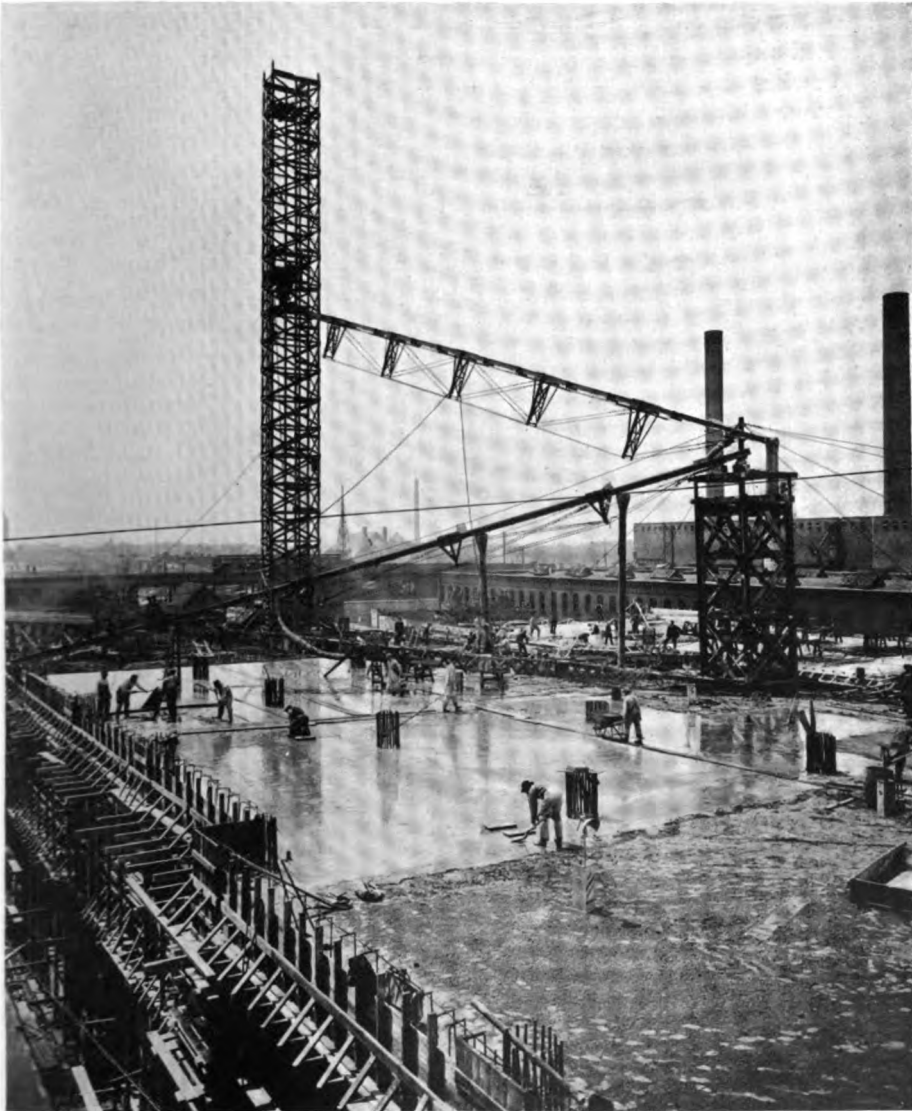


Fig. 7. View Showing Concrete Plant.
REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.

The spouting equipment presented some features of special interest which are shown in Fig. 7. In the first place, the chutes were rigid and self-supporting, so that there was no sagging in them. The chute consisted of a pair of 8-in. channels placed back to back, 12 in. apart, with bottom lacing and an

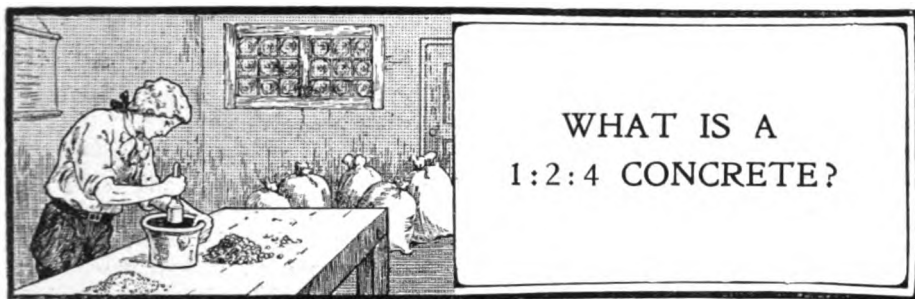
interior trough of sheet steel. This formed a chord member that was trussed from beneath by struts and cables. The main chute extended from the receiving hopper at the top of the elevator tower to a shorter tower near the centre of the building; from this second tower extended a second chute pivoted at the tower and supported by a latticed steel boom. The lower end of the boom was carried on a circular track around the tower, thus permitting the boom and chute to swing through a complete circle, reaching all parts of the floor.

An unusual feature was the use of flexible drop pipes, by which the concrete could be taken from several openings in the revolving chute and delivered to the forms. These vertical pipes were of metal, spaced 10 ft. apart and fitted with gates at the upper end which might be opened to draw off concrete from the chute.

The architect for the building was John Graham, of Detroit, supervising architect for the Ford Motor Co., and the structural design was made by the Condron Co., engineers, of Chicago. The general contractor was E. L. Schneidenhelm, of Chicago.



Fig. 8. Exterior View.
REINFORCED CONCRETE BUILDING FOR THE FORD MOTOR CO., CHICAGO.



By H. C. JOHNSON, Lecturer, University College, Cork.

The following article will probably interest all those who make a special study of the all-important questions of Concrete aggregates and the mixing of concrete.—ED.

In these days of scientific proportioning of materials for concrete more ink upon the subject seems absurd, but when it can be definitely stated that there is as much difference in the amounts of cement in the "popular" 1:2:4 concretes as there is between 100 bags of cement and 130 bags of cement it may be admitted that we have *new ink* upon the subject.

In order to have some basis upon which to consider this astonishing difference it will be necessary to state the *general* requirements of the great majority of 1:2:4 concrete specifications: *The concrete shall be composed of water, with one volume of cement, two volumes of fine aggregate (sand), and four volumes of large aggregate. The cement must pass the requirements of the British Standard Specification, while the others must be of good quality and clean.*

Now that specification does not look as if it would allow a contractor to save or lose 30 bags of cement—the difference between 100 bags and 130 bags—and still allow him to fulfil its demands, but it is so.

If that 24 per cent. can be saved by a contractor, it goes straight into his pocket, *plus the saving in not having to handle that quantity*; in other words, it is a *clear* profit, but a loss to the structure—a loss that seriously reduces its strength. This does not imply wickedness on the part of the contractor, but rather carelessness on the part of the architect or engineer.

The trouble is that the proportions 1:2:4, with too many people, stand for 2,000 lb. per sq. in. at 28 days. Some mixtures of 1:2:4 will allow that strength at that age—most will not come nearer than 1,500 lb. Such a statement does not condemn concrete—it condemns the makers.

For the present we will leave out the question of strength and consider only the amount, or, better still, the *volume percentage*, of cement in various 1:2:4 concretes. After all, the cement is the only thing that "cements," and the more of it, all other things being equal, the stronger the concrete, statements to the contrary notwithstanding.

Before going further the writer will assume that the reader will admit the truth of the following facts:—

(a) Sand, stone and gravel are inert materials with absolutely no cementing power, and are only used to *dilute* neat cement, because neat cement would be too expensive to use alone. [This is quite apart from the question of using neat cement alone, since no one would use neat cement so.]

(b) Aggregates possess voids, or air spaces, ranging from 25 per cent. to 50 per cent. of the gross volume they occupy.

(c) It is incorrect to speak of a 1:2:4 mix as a 1:6 mix, since it may convey the impression that 6 parts of sandy gravel with 1 of cement is the same as 4 parts stone and 2 parts sand with 1 of cement.

(d) It is generally considered that a 1:2:4 mix gives a final volume of slightly over 4 parts, so that one-quarter of the mass is cement.

The following case illustrates the trouble caused by a loose specification. The architect called for "concrete to be composed of 6 parts of stone and sand, both measured by loose volume, to 1 part of Portland cement . . . by volume." The contractor was fortunate enough to find on the site a gravel of good quality, and asked the architect's permission to use it, which was readily granted. The contractor proceeded to mix 6 parts of the gravel with 1 of cement, when the architect stopped him, pointing out that the gravel should first be screened to remove the sand and then remixed in the proportions of 2 parts sand to 4 parts gravel. The contractor claimed he had a right to use 6 parts to 1 of cement. Neither side would give in, and the case went up for arbitration, when both admitted faults—the architect that his specification should have read 1 volume cement, 2 volumes sand, and 4 volumes stone; the contractor that he thought, when he was allowed to use the gravel, that 6 parts to 1 of cement was equal to 4 stone, 2 sand, and 1 cement.

The actual percentage of cement in various 1:2:4 concretes.

No. 1.—We will suppose that the above contractor had his way, and proceed to find the actual percentage of cement in his concrete.

The percentage of voids in the usual run of bank gravel will be about 33. Six parts of this mixed with 1 part of cement by loose volume will produce 6 parts of finished concrete. But since cement (of average fineness) loosely measured reduces to about 70 per cent. of the original volume upon wetting there will be only $\frac{70}{100}$ of 1 part of cement to 6 finished volumes of concrete. Ratio 1 to 8.6 = 11.6 per cent. cement; weight per cu. ft. = 118 lb. [*Also note that of the 2 parts (33 per cent.) void, only a trifle over one-third is filled.*]

No. 2.—If the gravel is passed over the sand ($\frac{1}{2}$ -in.) screen and also the $\frac{3}{4}$ -in. screen, a large aggregate with about 38 per cent. voids and a small aggregate (sand) with 40 per cent. voids will be obtained. These mixed 4 parts large aggregate, 2 parts small aggregate, 1 part cement, will produce about 5.6 parts of finished concrete. Ratio 1 to 8 = 12.5 per cent. cement; weight per cu. ft. = 140 lb.

No. 3.—If only material that passes the $\frac{1}{2}$ -in. screen and is retained on the $\frac{3}{4}$ -in. screen, and which will contain about 42½ per cent. voids, be used as large aggregate and a mixture of 4 parts large aggregate, 2 parts small aggregate (as before), 1 part cement, there will be produced 5½ parts finished concrete. Ratio 1 to 7.5 = 13.3 per cent.; weight per cu. ft. = 143 lb.

No. 4.—A crushed limestone passing the $\frac{3}{4}$ -in. screen and retained on the $\frac{1}{2}$ -in. screen will contain 45½ per cent. voids, and if mixed 4 parts large aggregate, 2 parts small aggregate (sand from gravel), 1 part cement, there will be produced 4.92 parts finished concrete. Ratio 1 to 7 = 14.25 per cent.; weight per cu. ft. = 143 lb.

No. 5.—A crushed limestone passing the $\frac{1}{2}$ -in. screen and retained on the $\frac{3}{4}$ -in. screen will contain 48½ per cent. voids, and if mixed as No. 4 will give nearly the same finished volume. Ratio 1 to 7 = 14.25 per cent.; weight per cu. ft. = 144 lb.

Now some engineers prefer a sand with the finer particles removed and a larger large aggregate; we will therefore make two more mixes, using such material.

No. 6.—A crushed limestone passing $\frac{3}{4}$ -in. screen and retained on the $\frac{1}{2}$ -in. screen will have about 48 per cent. voids (about same as No. 5 stone), and if mixed 4 parts large aggregate, 2 parts small aggregate (sand taken from the gravel and passing $\frac{1}{2}$ -in. screen, retained on $\frac{3}{16}$ -in. screen—44 per cent. voids), 1 part cement, there will be

produced 4.8 parts finished concrete. Ratio 1 to 6.85 = 14.6 per cent.; weight per cu. ft. = 149 lb.

No. 7.—A gravel sized as for last mix (No. 6) will contain 43 per cent. voids, and if mixed 4 parts large aggregate, 2 parts small aggregate (as for No. 6), 1 part cement, there will be produced 5.10 parts finished concrete. Ratio 1 to 7.3 = 13.7 per cent.; weight per cu. ft. = 144 lb.

Other engineers and architects prefer to make a mortar of the sand and cement in the ratio of 2 parts sand to 1 part cement, then use 2 parts of this mortar to 4 parts stone. With this method we will make two more concretes—one stone and one gravel.

No. 8.—Using the gravel passing the $\frac{3}{8}$ -in. screen and retained on the $\frac{1}{8}$ -in. screen, but with material from $\frac{1}{2}$ in. to $\frac{3}{8}$ in. removed, voids will equal 35½ per cent. By mixing 2 parts sand (obtained from gravel) passing $\frac{1}{8}$ -in. sieve, retained on $\frac{1}{16}$ -in. sieve, containing 40 per cent. voids, with 1 part cement, and then wetting these to make mortar, 2.64 parts mortar will be produced (a finer sand will produce more). These mixed, 4 parts large aggregate, 2 parts mortar, will produce 4.96 parts finished concrete. In order to obtain the ratio it must be remembered that 1 part cement is contained in the 2.64 parts mortar; therefore by using only 2 of the 2.64 parts of mortar only 76 per cent. of the cement is in the concrete. Ratio = $\frac{2 \times 76}{4.96} = 1$ to 9.35 = 10.7 per cent.; weight per cu. ft. = 142 lb.

No. 9.—Using crushed limestone of same sizes as above, which will have 45 per cent. voids, and also mortar of same proportions: if mixed 4 parts large aggregate, 2 parts mortar, there will be produced 4.4 parts of finished concrete. Ratio (worked as above) = 1 to 8.3 = 12 per cent.; weight per cu. ft. = 145 lb.

Returning now to *true* 1 : 2 : 4 concretes (the last two are not):—

No. 10.—Using crushed stone passing $\frac{3}{8}$ -in. screen and retained on $\frac{1}{8}$ -in. screen, and which will have 46 per cent. voids, and a very good quality but very fine sand passing $\frac{1}{8}$ -in. sieve and retained on $\frac{1}{16}$ -in. sieve, and making a concrete with these—4 parts large aggregate, 2 parts small aggregate, 1 part cement—there will be produced 4.33 parts finished concrete. Ratio 1 to 6.2 = 16.1 per cent.; weight per cu. ft. = 144 lb.

The following four mixes clearly show that a fine sand produces more, but a weaker, concrete than coarse sand:—

No. 11.—A $\frac{3}{8}$ -in. to $\frac{1}{2}$ -in. washed gravel, with a sand $\frac{1}{8}$ -in. to $\frac{1}{16}$ -in., but in which the *coarse* particles predominate (same sand as used in Nos. 1 to 9 inclusive, but carefully washed)—4 parts large aggregate, 2 parts small aggregate, 1 part cement—produces 5.26 parts finished concrete. Ratio 1 to 7.51 = 13.3 per cent.; weight per cu. ft. = 144 lb.

No. 12.—Washed gravel as above, sand *same* sieves as above, but one in which the *fine* particles predominate—4 parts large aggregate, 2 parts small aggregate, 1 part cement—produces 5.38 parts finished concrete. Ratio 1 to 7.7 = 12.96 per cent.; weight per cu. ft. = 146½ lb.

No. 13.—A washed crushed limestone, same size as No. 11 and same sand—4 parts large aggregate, 2 parts small aggregate, 1 part cement—produces 4.83 parts finished concrete. Ratio 1 to 6.9 = 14.48 per cent.; weight per cu. ft. = 148½ lb.

No. 14.—Washed limestone same as for No. 13, and sand same as used in No. 12—4 parts large aggregate, 2 parts small aggregate, 1 part cement—produces 5.0 parts finished concrete. Ratio 1 to 7.14 = 14 per cent.; weight per cu. ft. = 149½ lb.

To this list might be added six other mixtures just as carefully made by the writer as those above, but since we have, in the ones given, the maximum and minimum percentages of cement found to be in the twenty 1 : 2 : 4 mixes made, no useful purpose, under the present title, would be served by including them, since they either give the same percentages, or nearly the same, as those listed.

No. of Mix.	Large Aggregate and per cent. Voids.	Small Aggregate and per cent. Voids.	Loose Volumes.			Finished Volumes.	Weight per cu. ft. Wet.	Cement in Finished Volume of Concrete.	
			Large Aggregate.	Small.	Cement.			By Ratio.	By per cent.
1	Gravel as from river (33 per cent.)	None	6	—	1	6.0	lb. 118	1 to 8.6	11.60
2	Gravel, $\frac{3}{4}$ "-1" (38 per cent.)	Sand from gravel, $\frac{3}{8}$ "- $\frac{7}{8}$ " (40 per cent.)	4	2	1	5.6	140	1 " 8.0	12.50
3	Gravel, $\frac{1}{2}$ "- $\frac{3}{4}$ " (42½ per cent.)	Ditto	4	2	1	5.25	143	1 " 7.5	13.30
7	Gravel, $\frac{3}{8}$ "- $\frac{1}{2}$ " (43 per cent.)	Sand from gravel, $\frac{3}{8}$ "- $\frac{3}{4}$ " (44 per cent.)	4	2	1	5.10	144	1 " 7.3	13.70
8	Gravel, $\frac{3}{8}$ "-1", less $\frac{1}{4}$ "- $\frac{3}{8}$ " material (35½ per cent.)	Sand from gravel, $\frac{3}{8}$ "- $\frac{7}{8}$ " (40 per cent.)	4	(2 parts (2-1 (mortar	)	4.96	142	1 " 9.35	10.70
11	Washed gravel, $\frac{3}{4}$ "-1" (34½ per cent.)	Sand from gravel carefully washed, $\frac{3}{8}$ "- $\frac{7}{8}$ " (42 per cent.)	4	2	1	5.26	144	1 " 7.51	13.30
12	Ditto	Special sand quite clean, $\frac{3}{8}$ "- $\frac{7}{8}$ " (35 per cent.)	4	2	1	5.38	146½	1 " 7.7	12.96
4	Crushed limestone, size as No. 2 (45½ per cent.)	Sand as for No. 2	4	2	1	4.92	143	1 " 7.0	14.25
5	Crushed limestone, size as No. 3 (48½ per cent.)	Ditto	4	2	1	4.90	144	1 " 7.0	14.25
6	Crushed limestone, size as No. 7 (48 per cent.)	Sand as for No. 7	4	2	1	4.80	149	1 " 6.85	14.60
9	Crushed limestone, size as No. 8 (45 per cent.)	Sand as for No. 8	4	(2 parts (2-1 (mortar	)	4.4	145	1 " 8.3	12.00
13	Washed crushed limestone, size as No. 11 (45 per cent.)	Sand as for No. 11	4	2	1	4.83	148½	1 " 6.9	14.48
4	Ditto	Sand as for No. 12	4	2	1	5.00	149½	1 " 7.14	14.00
10	Crushed whinstone, $\frac{3}{4}$ "-1" (46 per cent.)	Fine sand, $\frac{3}{8}$ "- $\frac{1}{2}$ " (48 per cent.)	4	2	1	4.33	144	1 " 6.2	16.10

Gravel Concretes (excepting Nos. 1 and 8).—Average per cent. cement = 13.15; average per cent. voids = 38½; average weight per cu. ft. = 143½ lb.

Stone Concretes (excepting Nos. 9 and 10).—Average per cent. cement = 14.32; average per cent. voids = 46½; average weight per cu. ft. = 146½ lb.

Figures like the above are generally more easily remembered if in the form of a table, and that given contains all the necessary information for the further consideration of the many interesting points these tests show.

It is very necessary to impress the reader with the fact that all measurements were made by *loose volume*—in the proportion 4 large aggregate, 2 small aggregate, to 1 cement—just as would be made on the job, the methods on which the writer is fully acquainted with. The reason for taking the liberty of stating the last-mentioned fact is that many are inclined to take laboratory work and laboratory suggestions with a grain of salt, or with an easy mind, so to speak. From where else, pray, do they obtain their safe value of 16,000 lb. per sq. in. for steel, or safe $\frac{l}{k}$ for columns but from the laboratory? To our laboratories, therefore, we must look for the *super-concrete*, and it is hoped that the present article may do its little towards that end.

In order to obtain uniformity in proportions, the weights per cubic foot of the sands and the cement (the same brand—Ship Canal Co.'s—being used for all mixes) were obtained at the start and used for mixing. All materials dry.

In summing up the points contained in the table no use will be made of Nos. 1 to 8 and 9 because they are not 1 : 2 : 4 concretes in the strict sense, although many have been guilty of so considering them—guilty because even departures from rule-of-thumb methods, which are often satisfactory, must not be made to satisfy theory without first proving theory to be correct practically. The table illustrates the following facts:—

(a) 1 : 2 : 4 concrete No. 10 contains 29 per cent. more cement than 1 : 2 : 4 concrete No. 2.

(b) Stone concretes on the average contain 10 per cent. more cement than gravel concretes. It is therefore not fair to expect gravel concretes to be as strong as stone concretes. ("American Civil Engineer's Pocketbook" notes that gravel concretes at one year are 9 per cent. weaker than stone concretes.)

(c) The percentage of voids in gravel being smaller than in stone, less sand should be used: $1\frac{1}{2}$ parts sand (on the average) to 4 parts gravel will produce a concrete worked as easily as, if not more easily than, 2 sand to 4 stone.

(d) Gravels passed through and retained on screens as used for stone *always* have less voids; the popular idea is that they would be the same. It can be admitted that if stone could be "hand placed" into a receptacle, a very small percentage of voids would result. This indicates that stone concretes, unless they had considerable sand (2 to 4), would not be so reliable as gravel in the matter of density.

(e) Gravel concretes mixed 1 : 2 : 4 are 3 lb. per cu. ft. lighter than stone concretes. Proper proportioning will easily make up this 3 lb., in spite of the fact that the specific gravity of gravel is only 2.51, as against 2.70 for stone.

The writer has used the terms *large aggregate* and *small aggregate* on purpose to draw attention to the fact that sand is only finer gravel or granite or quartz, is an inert material just the same as the larger materials, and has to be "cemented" by the cement just as the larger particles. It is no use talking of mortar filling the voids or anything of that sort. Mortar is used by bricklayers and masons to bed two plane surfaces together; concretors use inert aggregate and cement with which to bind it together; but the outstanding fact is that the *smaller the aggregate* (all other things being equal) the *weaker the concrete*. There is no need to quote authorities; they are many, and have established the fact. In view of this last, the less fine aggregate (sand), providing a thoroughly workable concrete is produced, the better.

This article contains no strengths of these mixes of 1 : 2 : 4 concretes, and this is not necessary under the title, but it was stated at the beginning that "the more cement

the stronger the concrete—“*statements to the contrary notwithstanding.*” Statements have been made, and tests devised to prove, that—

(a) Reducing the percentage of voids to the absolute minimum possible, and filling these with cement, produces the strongest concrete.

(b) Any cement beyond that necessary to fill these voids is waste and does not increase the strength.

(c) Just as a carpenter uses the minimum of glue in making a joint, so one should use a minimum of cement in making concrete.

Regarding (a), it is possible to reduce the percentage of voids to 10 by using a graded sand, but 10 per cent. of cement has to coat the surface of the remaining 90 per cent. Could this be the strongest possible concrete?

Regarding (a) and (b), a recent paper sought to prove that a mix of 1 : 2 : 4 was not stronger than one of 1 : 3 : 7. It was *not* at one month, but it was *over 50 per cent. stronger* at three months, and at a year would probably leave a greater gap. In the one-month test the 1 : 2 : 4 concrete contained more of, what was then, the weak material than did the 1 : 3 : 7, but at three months it began to assert its value, with the result stated.

Regarding (c), there is no sense in this argument, since the carpenter, like the mason, is only sticking together two plane surfaces (in a tenon, several), and would therefore not obtain greater strength any more than an engine could pull more by using a longer coupling-chain.

Such statements would suggest that neat cement in compression would not be so strong as a mixture of very strong stone and cement. Two tests made by the writer can be put forward as against this idea. A first-class neat cement cube gave 12,064 lb. per sq. in. at *three months*. Another cube made of hard whinstone (probable crushing strength 20,000 lb. per sq. in.) $2\frac{1}{2}$ parts, and the same cement as above to fill the voids, one part, stood only 7,300 lb. per sq. in. at *eighteen months*.

It may be admitted, therefore, all other things being equal, the more cement the stronger the concrete. Gravel concrete has suffered for years, because for *apparently* equal quantities of cement it is not as strong as stone concrete.

The whole trouble lies in the fact that the amount of cement is not based on the finished volume of concrete.

Now, in suggesting remedies or corrections, other investigators go too minutely into proportions, and therefore do not succeed in carrying the “practical” man with them in their ideas; but the following method, not entirely new (Mr. W. B. Fuller years ago suggested a similar method, but gave no figures that the writer can find), is simple, and after once done can easily be remembered.

TO DETERMINE CORRECT PROPORTIONS.

Correct method of proportioning 1 : 2 : 4 concretes.—Apparatus required: 2 pails holding $\frac{1}{2}$ cu. ft. each, trowel, scales to weigh to 100 lb., materials to be used for the concrete, and water. Before mixing, first determine the percentage of cement to be used. For instance, using the figures given in the table, and considering that two engineers out of three prefer stone concrete, we will find an average for all the 1 : 2 : 4 mixes, thus:—

Average percentage cement in stone concretes	...	...	= 14'32
“ “ “ gravel “	...	...	= 13'15
If 2 engineers prefer stone	...	...	$2 \times 14'32 = 28'64$
and 1 engineer prefers gravel	...	...	$1 \times 13'15 = 13'15$
			3 41'79

Average = 13'93 per cent.

or, say, 14 per cent. compact cement = 20 per cent. loose volume.

Take one pail and measure 40 cu. ft. large aggregate and turn this on to mixing board, then measure 10 cu. ft. cement carefully. If large aggregate is gravel, measure out 125 cu. ft. sand (if stone, 15 cu. ft.) and turn these on to mixing board and mix with water—fill into other pail, noting volume and weight. This will not usually produce $\frac{1}{2}$ cu. ft. or a pailful (which is necessary to obtain the correct percentage for the amount of cement used), so continue adding sand in small lots carefully measured until a *working consistency* is obtained, after which weigh and note volume; if pail is not full, add large aggregate, noting the volume, until pail is full. The last figures obtained give the proportions for the mix.

EXAMPLE.

	Volume.	Weight. lb.	Weight per cu. ft.
Gravel 4 cu. ft. + 1 cement + 125 sand ...	435	62	144
.. 4 .. + 1 .. + 15	460	67	145½
.. 4 .. + 1 .. + 175	488	71	145½
<i>(This produces a workable concrete, therefore add more stone.)</i>			
.. 425 .. + 1 .. + 175	502	73½	146½

N.B.—No account need be taken of the water used, but a trifle more than sufficient should be used in first batch and none added after.

The correct mix, then, is 425 large aggregate = $4\frac{1}{4}$, 175 small aggregate = $1\frac{3}{4}$, 100 cement = 1. Of course, these figures will only apply to the concrete using this sand. Another sand, with the same gravel, might produce the proportions 1 : 1·6 : 4, but there is still the same expenditure of cement for the same *finished* volume of concrete. Smaller proportions do not indicate smaller finished volumes.

Using this method, the two following concretes were produced, using the 14 per cent. wetted cement = 20 per cent. by loose volume :—

A.—Gravel $4\frac{1}{4}$ parts, fine sand $1\frac{1}{2}$ parts, cement 1 part. Weight per cu. ft., 148 lb.

B.—Same gravel 4 parts, coarse sand 1·6 parts, cement 1 part. Weight per cu. ft., 146 lb.

Two others, using stone and the above sands, called for proportions so near 1 : 2 : 4 (one was No. 14 in table) that nothing would be gained by altering those proportions. This would not prove that stone concretes need not be proportioned, since only one size stone was used in these two mixes.

Correct specification for concrete equal to average 1 : 2 : 4 concrete—i.e., containing 20 per cent. by loose volume of dry cement equal to 14 per cent. wetted :—

The concrete shall be composed of perfectly clean crushed stone or gravel passing the $\frac{1}{2}$ -in. sieve and retained on the $\frac{3}{8}$ -in. sieve ($\frac{1}{4}$ -in. suggested); the particles of gravel shall have a least dimension of not less than one-half the greatest dimension, the stone a least dimension not less than one-third the greatest dimension, and no flakes that can be levered with a knife from the surface of any of the particles. Specific gravity not less than 2·5.

The sand shall be thoroughly washed, and when dry shall pass the $\frac{1}{4}$ -in. sieve and be retained on the $\frac{3}{8}$ -in. sieve, the particles shall be as nearly spherical as those of the gravel mentioned above. Specific gravity not less than 2·5.

The cement shall *easily* pass the minimum requirements of the British Standard Specifications of August, 1910, when not less than 22 per cent. of water is used for neat cement specimens and not less than 8½ per cent. for mortar specimens.

The proportions for mixing shall be determined on the principle that the minimum amount of sand consistent with easy working of the mass shall be introduced (not more than will give a ratio of 1 sand to 2 stone for stone concrete, or not more than

will give a ratio of 1 sand to 2.4 for gravel concretes), and that the resultant thoroughly mixed concrete shall not contain less than 20 per cent. of cement by loose volume or 14 per cent. by wetted (22 per cent. water) volume. The weight per cu. ft. when wet shall not be less than 145 lb.

Samples of the materials proposed to be used shall be deposited with the undersigned (architect or engineer) before any concrete materials arrive on the job, and the proportions to be used shall be proved to be as called for, by a demonstration in the presence of the undersigned, who will then give written authority to use such proportions if found correct.

The latter part of this specification is strictly in keeping with the sense of this article. The first part is not, but is nevertheless true and necessary to a complete specification, and is therefore included.

The specimens made for this article will be tested in due course, and the results given, when the effect of a rational method of proportioning will show itself.

The writer, in conclusion, appeals to other investigators to give, in future, the percentage of cement per *finished volume* of any concretes they may report upon, as being the best way to compare strengths for given expenditures in cement.



AN INSTANCE OF LIGHTNING ON A STEEL DOME AND REINFORCED CONCRETE WALLS.

The following particulars regarding the effect of lightning on reinforced concrete are the result of some observations made by Mr. C. D. Perrine of the Argentine National Observatory at Cordoba. He first reported on them in a letter published in "Science," and afterwards reprinted in the "Engineering News."—ED.

It is evident, says the *Engineering News*, that the domes and reinforcing were carefully grounded to lead lightning discharges safely to ground. Freedom from disturbances within the domes was expected, as the construction was most favourable for such protection. Mr. Perrine's report reads as follows:—

The centre of the storm, judging from the clouds and their motions, was not over a mile south by south-east of the observatory. In nearly all the storms which I had seen here previously the discharges were nearly all between clouds. (Perhaps because most of them occur at night?) In this storm nearly all of the discharges were between the clouds and earth.

The direction of motion of this storm, as is usually the case, was from south to north. After some half dozen discharges close to the south there was a heavy one to the north-west about three hundred metres away—then another to the north-east about the same distance.

On account of this being a heavy storm and apparently passing directly over us, I was interested to see what the effect would be on our two new reinforced concrete walls and steel domes sheathed with galvanised iron, and was outside among the central group of buildings and not over 100 ft. from the domes in question, one of them in full sight.

A minute or two after the flash to the north-east, mentioned above, there was a general illumination close by, followed almost instantly by the ripping sound of a very close stroke. The interval between the flash and the sound was certainly not over one-tenth of a second. To me the sound appeared to be made up of three or four separate discharges blended into one—not consecutive.

The power and light currents were cut off until about 6 p.m., when it was found that fuses had been blown on our lines (which were special ones) just outside the step-down-station, some 400 metres away. No other effects of the storm were noticed in or near this station.

The dome, which had just been completed, was barely out of sight from where I stood, and no one at the observatory seems to have seen the actual flash. A peon, however, in the grounds of the Meteorological office, about 100 metres away, had a full view of both domes and buildings, was facing them and saw the flash just over and about the new dome. This accords well with the direction and distance from my point of observation.

After hearing of this observation I made a careful examination of the dome, and in particular the connection of the copper cable with the track upon which the dome revolves, which forms the connection between the metal dome and one of the vertical I-beams embedded in the concrete for grounding the circuit. The lightning-rod proper extends about a metre above the highest part of the dome and terminates in a brush of heavy wire. No signs whatever of any discharge have been found at any point about the dome.

Close to the dome stands the wooden derrick which was used in its construction, the top of which is about 2 ft. higher above the ground than the lightning-rod. Three wire-cable guys lead off to trees; two of them actually touch the ground—but scarcely so—and a fourth goes to a brick building. The cable used for lifting did not touch the ground. Careful examination of all of these points failed also to disclose the slightest sign of a spark.

The three wires of the alternating power circuit pass close to both dome and derrick.

The bolt which struck the dome was undoubtedly not a light one, for it frightened badly a number of persons in the residences near by, and was described by several as a very bright flash. I do not think, however, that it was an especially heavy one, possibly not so heavy as most of the others which struck in the vicinity.

The peon who saw it from the neighbouring quinta was seated at the time under a shed and watching the dome. He says the flash appeared to descend as a single ray, striking the lightning rod and then the whole surface of the metallic dome appeared to be covered with sparks or flashes.

It seems certain, therefore, that the dome was actually the principal point of discharge for a fairly heavy flash of lightning. (It is uncertain how much of the discharge was taken by the derrick, but it would appear to have been relatively small.) Also the induced currents in the light and power lines were sufficiently heavy to blow the fuses in both.

At the time the bolt struck, there was a peon inside the closed dome, cleaning the running-gear. When questioned he said he had felt nothing nor had he noticed anything unusual beyond the heavy noise.

This experience seems to be a fairly severe test for such a construction—a metallic dome surmounting concrete walls which are heavily reinforced with iron—the metal in the walls having a good ground connection and being connected also with the dome.



*It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.
The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.*

THE CONCRETE INSTITUTE.

THE APPLICATION OF CONCRETE IN MODERN SANITATION.

By HENRY J. TINGLE, M.Inst.C.E.

An exceedingly interesting and instructive Paper was read at the meeting of the Concrete Institute held on January 7th on "The Application of Concrete in Modern Sanitation." The Paper was illustrated by photographs and diagrams, and, in addition, there was an Appendix dealing with the manufacture of concrete tubes.

We give below an abstract from the Paper, together with a short résumé of the discussion which followed.

INTRODUCTION.

THE term "Sanitation" is capable of a wide interpretation, but for the present purpose it is proposed to limit its definition to Works of Water Supply, Main Sewerage, and Sewage Disposal.

Concrete has played its part in sanitation for many years, with more or less success, according to the skill of the designer and the quality of the materials and workmanship. The tendency in recent years has been to employ concrete more and more, and this tendency in the future is certain to extend its use in directions where now it is scarcely applied.

There is no doubt that sanitation works have been peculiarly affected by the attitude of the Local Government Board towards reinforced concrete, as such works are generally undertaken by local authorities, who have to obtain the sanction of the Board to the raising of a loan for the amount of capital expenditure, or else to obtain Parliamentary powers by means of a Private Act.

Doubtless in the early years of reinforced concrete the Board were well advised to act cautiously in the matter of sanctioning loans for the use of a combination of materials of which, in this country, there had been scarcely any experience, either as to its suitability or as to its life in sanitation works.

With the experience that has now been acquired relating to the behaviour of reinforced concrete that is constantly in contact either with water or with sewage, it is permissible to hope that the time is not far distant when the Local Government Board will reconsider its attitude towards this material, and in cases where it is proposed to make use of it will sanction loans on more favourable terms as to the period of repayment than those now in force.

Concrete has been and now is extensively employed in the construction of reservoirs, sewage tanks, filters, aqueducts, and water towers.

SEWERAGE WORK.

Mass concrete has been largely used in the construction of sewers of 4 ft. diameter and upwards. It is moulded in the trench and supported by centering until set. Elliptical sewers are also constructed in this manner. In recent years reinforced concrete has been employed for sewers of both types, the mixture being sufficiently stiff to admit of ramming.

Concrete tubes have been extensively employed in main sewerage work for many years past, being supplied ready for use by firms who have taken up their manufacture.

Concrete tubes, unless strongly reinforced, should be laid on a concrete bed not less than 6 in. in thickness, and the concrete be brought up round the tube as far as the springing. By the omission of this precaution many failures have occurred. In tubes liable to internal pressure the concrete should entirely surround them.

The tubes have ogee, rebated joints, which are luted with cement and the inside pointed up.

In a paper read by Mr. E. J. Elford, M.Inst.C.E., on June 6th, 1914, before the Institution of Municipal and County Engineers (*vide* Volume XL. of *Proceedings*), he describes the method of jointing concrete tubes laid in tunnel at Southend-on-Sea. The following extract from his paper gives the particulars:—

"The socket and spigot were so designed as to provide when placed together an annular space within the joint of about $\frac{1}{4}$ in., and the specification required the contractor to lay the tubes with dry, clean joints, fill in solidly with concrete on the outside, and afterwards to make the joints by first sealing the interior with an expanding steel and rubber ring, and then forcing liquid cement into the annular space through a hole in the ring at the side or invert. A second hole was to be provided at the top of the ring as an air vent, and this was to be left open until the cement began to flow out, after which it was to be plugged and the cement forced in until the joint was solid."

The joints were quite satisfactory.

Concrete tubes are manufactured on the "Jagger" system without ramming the matrix. The moulds are placed on a table to which a motion is imparted, described as "a horizontal vibratory movement combined with a vertical oscillation accompanied by a suddenly arrested rocking movement which might be expressed as intermittent gliding shocks in quick rotation." The mixture is fed into the moulds, and the effect of the movements of the table is to cause the aggregates to settle down into a compact mass, without stratification, before the initial set of the cement occurs. It is also claimed that all voids disappear. The presence of reinforcement does not interfere with the solidification of the concrete.

In bad ground it is advisable to strengthen the concrete under the tubes by inserting reinforcement therein, thus forming a raft foundation.

In some instances artificial means are adopted to hasten the setting of the concrete while the tube is in the mould by exposure to hot air, with a view of releasing the moulds more quickly. It is claimed that the precautions taken during this process prevent any weakening of the resisting power of the concrete. It would appear that this is a case where investigation is required to ascertain more definite information.

For sewers, the tubes are usually 3 ft. in length, and are sometimes slightly reinforced by the insertion of three rings of steel $\frac{1}{4}$ in. diameter, spaced about 12 in. apart. The joints in the rings are welded. There are no longitudinal nor spiral rods. It is found that this simple type of reinforcement gives to the tube increased resistance to crushing.

Tubes of the above type are very suitable for sewers, but are not adapted to withstand high internal pressure nor shock from moving columns of water.

Tubes are made capable of withstanding high internal pressure, and of resisting water hammer. They are constructed with various systems of reinforcement, and are in use in Great Britain and more generally abroad. It is not possible to refer to all the systems of reinforcement adopted. Generally they consist of various combinations of helicals and longitudinals, with the addition in some instances of circulars.

The "Bonna" system goes a step farther, and is essentially a thin steel pipe strengthened and protected, externally and internally, by reinforced concrete. The

internal reinforcement is not taken into consideration in the stress calculations, but its presence is necessary to support the internal concrete. The dimensions of the cruciform steel bar forming the helicals, and the distances of the helicals from one another, together with the thickness of the concrete, vary according to the diameter of the pipe and the pressure to be resisted.

The pipes are frequently manufactured on the site of the works where they are to be laid. They are usually 10 ft. in length, and have been made from 10 to 78 in. in diameter. In one instance, at Rio Soza, in Spain, two syphons, each 12 ft. 6 in. diameter, were laid under two canals.

The "Siegwart" reinforced pipes are constructed as follows: A steel core is revolved whilst concrete is applied by machinery until the required thickness is obtained. Steel wire is then wound by the same machine spirally over the concrete, in one or more layers as required. The assembled longitudinal rods are slipped over the spirals, and further concrete applied by the machine until the pipe is finished.

These pipes are made 15 ft. long for diameters 12 in. to 18 in., and 12 ft. long for larger diameters. They have been used for water mains in Cairo, Zurich, Italy, and as a pumping main at Grays, Essex, the pressures being from 20 lb. to 100 lb. per sq. in., and have been tested at the Grays works to 300 lb. per sq. in.

Both socket and spigot pipes, and butt-end pipes, with reinforced collars are made. Concrete tubes used as sewers may suffer damage from liquids being discharged into them at high temperatures, also by the discharge of acids. Instances have been recorded where concrete has perished when in contact with sewage.

In the great majority of cases concrete suffers no damage by contact with sewage. Still, under certain conditions the lime in the cement may be attacked, and, as far as is possible, precautions should be taken to prevent such conditions arising. Further investigation is needed to ascertain why concrete occasionally disintegrates when in contact with sewage.

MANHOLES, WATER TANKS, AND CESSPITS.

Concrete tubes are constructed to form manholes, water-tanks, and cesspits.

Messrs. Sharp, Jones adopt the following method for manholes: The foundation block is of concrete, made in one piece, with the inverts moulded, for the main sewer and the branches joining the same. It is lowered into the trench by the aid of ring bolts, then the tubes forming the chamber are added, and as many feet of tubes forming the shaft as are required. Foot irons are provided. The shaft is covered with a special block of concrete, which carries the iron cover and frame.

Sewers, reservoirs, manholes, etc., are constructed with moulded blocks of concrete which are supplied by various manufacturers.

The Dean Stone Co., of Devonport, manufacture blocks from crushed syenite obtained from the company's quarries at St. Keverne, Cornwall, and Portland cement.

Straight and angle blocks are made for reservoir construction. The blocks are $4\frac{1}{2}$ in. thick and have $1\frac{1}{2}$ in. hole and frog. They are 1 ft. in height, and of varying lengths to admit of breaking joint at each course. The bottom course is made stronger, the blocks radiating from $4\frac{1}{2}$ in. thick at the top to 9 in. at the bottom, with a special 4 in. base, with groove to pin in the floor paving blocks. Liquid grout, composed of one part cement to one part of fine-sifted crushed syenite, is, when the wall coursing is in position, poured down the $1\frac{1}{2}$ in. hole, which it fills together with the frog underneath.

Concrete blocks are also made by the above-mentioned firm to provide a bearing under each joint of an ordinary stoneware pipe sewer. When the pipes are in position, liquid cement is poured into the opening left between the outside of the pipe collar and the block, until the cement stands nearly level with the upper surface of the block; by this means a good bottom joint is made in the sewer.

The following instances within the author's experience are given as illustrating, in greater detail, the application of concrete to sanitation works:—

Failure and reconstruction of concrete-covered reservoir.

Temperature cracks in concrete reservoir wall.

Ejector chamber in concrete with steel-plate lining.

Concrete mole forming sewage outfall into sea.

Circular hydrolytic tanks in reinforced concrete.

FAILURE AND RECONSTRUCTION OF CONCRETE-COVERED RESERVOIR.

This reservoir was constructed in concrete in the year 1889, and stood for two years when the failure occurred. The internal dimensions were:—

East to west	...	...	...	112 ft. 3 in.
North to south	...	...	...	79 ft. 6 in.
Capacity	...	...	...	One million gallons.
Level of invert of overflow	...	...	...	18 ft. 10 in. above floor.

The reservoir was divided into two parts by a longitudinal central division wall from east to west, and into five bays by four transverse walls from north to south; all the interior walls had arched openings. The bays were roofed in by concrete arches springing from the east and west walls and the transverse walls.

Ample ventilation was provided by ten ventilators in the roof, each 18 in. in diameter. The roof was covered with soil to a depth of 2 ft. over the crown of the central bay, and 1 ft. 6 in. over the outside bays.

The reservoir was partly in and partly out of ground, the floor line being about 11 ft. below ground level. The sub-soil was clay, with a pot-hole of gravel in the east bay, in which was a weak spring of water. Earth banks, with slopes varying from $1\frac{1}{2}$ to 1, to 1 to 1, were placed round the walls where out of ground.

The walls were without counterforts. The south wall was strengthened by the addition of three valve chambers.

The floor was 2 ft. in thickness throughout.

The thickness of concrete in the arches was 1 ft. at the crown and 1 ft. 3 in. at 2 ft. from the springing.

Water was pumped into the reservoir through a rising main 10 in. in diameter.

The concrete used in the work was composed of Thames ballast and Portland cement specified 6 to 1 mixture. It was, on examination after the failure, found to be dense, hard, and well set, except in the part of the floor over the pot-hole, and great difficulty was experienced when the repairs were effected in cutting the benchings in the undamaged portions of the walls to receive the new work. The interior surfaces of the exterior walls were rendered in cement.

For some days prior to, and on the day of the failure, continuous pumping into the reservoir had been in operation.

The reservoir collapsed suddenly without any previous signs of weakness having been detected, the west wall breaking away from the arch, the north and south walls, and the central interior wall.

The failure was due to faulty design, coupled possibly with over-pumping.

The repairs consisted of reconstructing the east and west walls in brickwork and cement stepped into the undamaged lower portions of the original concrete, with the addition of concrete counterforts extending the full depth of the walls. Counterforts were also put into the north and south walls. The interior walls, where fractured, were reconstructed in brickwork, and brickwork rings, supported on brick piers, turned under the arches over the openings. The east and west roof arches were reconstructed in brickwork and given 4 ft. rise. The adjoining arch to the west arch was strengthened by five brick arches turned under it. The concrete over the pot-hole was taken out and fresh concrete put in, the spring being drained clear of the site. The slag layer was removed. The overflow was reconstructed with easy bends, and an electric tell-tale connected in the pump-house.

Since reconstruction the reservoir has been, and now is, in continuous use, without needing repair.

TEMPERATURE CRACKS IN CONCRETE RESERVOIR WALL, HADHAM.

This reservoir was constructed in the year 1905. Depth of water, 11 ft. Subsoil, boulder clay. The floor of the reservoir was 6 ft. below ground level, and the roof and sides were covered by earth, except for a length of 10 ft. on the south-east wall, where the outer face of the wall was exposed to allow of access to the valve chamber, which was sunk into the ground and extended outwards from the lower portion of the reservoir wall for a distance of 7 ft. The chamber was covered with York stone about 6 in. above ground level, therefore for this length of 10 ft. the reservoir wall, for 8 ft.

deep measured from the top, was exposed to the atmosphere, and for the bottom 5 ft. was protected somewhat by the stone covering to the chamber.

The reservoir was entirely in concrete, except that brick piers were used to carry the rolled joists supporting the roof arches, and these arches had a $4\frac{1}{2}$ in. ring of brickwork under the concrete.

The concrete was 6 to 1 by measure of Thames ballast and Portland cement. The materials were turned twice dry and three times wet. The walls were 3 ft. 6 in. thick at the bottom and 1 ft. 8 in. at the springing of the roof arches.

The interior faces of the walls were rendered.

In the latter part of August, 1911, it was reported cracks had developed in the south-east wall both inside and outside the reservoir. On examination it was found that the rendering on the inside face was cracked in several places along its entire length, and that small cracks had formed in the concrete on the outer face where exposed. Also the portions of the rendering on the north-east and south-west walls adjoining the south-east wall were cracked. On cutting out the cracks on the outer exposed face they were found to penetrate into the concrete about $1\frac{1}{2}$ in., and those inside about 2 in. No signs of settlement could be detected by levelling.

In October the cracks were cut out and made good, and have not since reappeared.

EJECTOR CHAMBER IN CONCRETE WITH STEEL-PLATE LINING, SEAFORD.

This chamber was constructed on low-lying land situated 200 yards from the sea. The sub-soil was rock chalk, which as the excavation proceeded was found to be fissured, the sub-soil water being in connection with the sea.

The original intention was that the chamber should be square on plan and of larger dimensions than the circular one actually constructed, hence the excavation was taken out rectangular, 10 ft. by 16 ft., the depth being 16 ft. below ground level.

The construction of the chamber was carried out in the following manner: The bottom of the excavation was covered by ballast 1 ft. in thickness to allow the water to reach the sump, which was sunk outside the excavation. Planks 10 in. by 5 in. were placed 2 ft. 9 in. centres in the ballast, so that their upper faces were level with the top of the ballast. On these was laid a close-boarded platform 15 ft. by 15 ft., composed of 9 in. by 3 in. deals. This platform was strutted down to resist the water pressure underneath until sufficient concrete was laid to resist the same.

The circular tank rested on the platform, and was composed of riveted plates $\frac{3}{8}$ in. thick, stiffened by angle and T-irons; the internal diameter being 12 ft. 9 in. and the height 8 ft. 6 in. The tank arrived on the work in six segments, which were bolted up *in situ*.

The bottom halves to which the side walls for a height of 1 ft. 6 in. were riveted were placed in position and bolted up; then the remaining segments were bolted on, bag concrete being brought up round the outside to the level of -- 2'6 ft., after which mass concrete was continued up to the top of the chamber, the minimum thickness of the surrounding concrete being 2 ft.

The bottom plate of the tank was covered by concrete 2 ft. thick, in which two cast-iron trays were embedded, on which the ejectors were placed in order that their weight might be distributed. The interior of the tank was lined with 14 in. brickwork. A collar joint of 2 in. of cement was formed over the bottom and the sides, and the tank was covered with reinforced concrete, in which a manhole was provided for access to the chamber.

The concrete was composed of 200 lb. cement to $5\frac{1}{2}$ cu. ft. of sand and 11 cu. ft. of ballast, or 1 : $2\frac{1}{2}$: 5 mixture.

The work was satisfactorily completed in six weeks after the segments of the tank were delivered on the ground.

CONCRETE MOLE FORMING SEWAGE OUTFALL INTO SEA, SEAFORD.

This outfall consisted of a concrete mole in which two lines of cast-iron pipes were embedded. The site was an open bay in the English Channel, exposed to the full force of the south-westerly and south-easterly gales.

For a distance of from 80 yds. to 100 yds. from the sea wall the foreshore is covered with beach, and beyond this distance with a layer of sand about 2 ft. in thickness, under which is rock chalk.

An outfall of 18-in. pipes had been in existence at this spot for many years, and extended originally for a length of 83 yds. In 1901 it was carried a further 67 yds. by 24-in. cast-iron pipes. The pipes were held in position by oak piles driven into the solid chalk some 3 ft. deep, with oak cross-pieces to which the pipes were slung by iron straps.

In the spring of 1909 the travel of the beach over the top of the 18-in. pipe had worn a hole in it, through which beach gained access to the inside of the pipe, blocking it up. The head of sewage thus engendered burst the weakened pipe.

This burst took place at 81 yds. from the sea wall, where the beach ends and the sand is uncovered. Temporary repairs were effected and the pipes cleared.

Application had been made to the Local Government Board to re-sewer the town, and to extend and duplicate the outfall.

The contract for the work was commenced in the autumn of 1910, but for some time very little progress was made with the more difficult portion of the outfall.

The work was done as tide work at spring tides, and even then it was not possible to carry it on unless the sea was calm; in addition considerable damage to plant and concrete was occasioned by gales. The outfall was completed in August, 1913.

The whole outfall has stood the winter gales well, the concrete being sound. In places several bags were damaged, but have been made good.

The concrete was 1 : 2 : 4 mixture, mixed dry by hand, placed in the work dry, and wetted by the percolation of the sea water. The cement was burnt in rotatory kilns, ground all to pass 76 x 76 mesh, and residue on 180 x 180 not to exceed 10 per cent.; initial set not under 30 minutes, final set not under 5 hours nor over 10 hours.

Although not connected with concrete, it may be of interest to mention that the 2-in. deals used in the timbering round the concrete were attacked by teredo worm, and rendered utterly useless in under 12 months' exposure.

CIRCULAR HYDROLYTIC TANKS IN REINFORCED CONCRETE.

These tanks are designed and are about to be constructed for dealing with sewage on the Travis Hydrolytic Tank system.

The reinforcement is "Indented" bars, the stresses being limited to 16,000 lb. per sq. in. The concrete is 1 : 2 : 4 mixture. The shingle or broken stone is specified to pass through $\frac{3}{4}$ in. square mesh, and retained on $\frac{3}{8}$ in. The cement is to be burnt in rotatory kilns and to comply with all requirements of the latest British Standard Specification for slow-setting Portland cement, the quantity of cement in the mixture to be determined by weight. The stresses in the concrete are limited to 600 lb., and in direct compression to 500 lb. per sq. in.

The tank is divided into various compartments by concentric walls, and by radial division walls.

DISCUSSION.

The President, in thanking Mr. Tingle for his Paper, said it was one that was likely to be extremely useful to any of their members engaged in municipal engineering.

The "Bonna" pipe referred to by the author was exposed to currents of hot air to hasten the setting, but the experience of most architects and engineers was that reinforced concrete required some protection from hot air, such as placing a moist surface over it, etc., and it would certainly appear that they were sacrificing the strength of the pipes in order to shorten the time.

A cruciform steel bar was employed in the "Siegwart" pipes for forming the helicals, but he preferred the round or square bars.

Anyone who was in the habit of designing arches and abutment walls would at once discern the reason for the failure of the concrete-covered reservoir; there was not sufficient resistance in the abutting walls—they were bound to fail. In his opinion, when filling in the earth covering over reservoir arches, it was desirable, as far as possible, to begin at the ends of the arches and carry it carefully along, so that the thrusts were balanced all the way through and a uniform loading was obtained.

Mr. T. J. Moss-Flower, Assoc M.Inst.C.E., thought that the ejector chamber which was put down on the sea-shore was a rather expensive method of construction, and suggested that it would have been cheaper and more expeditious to have put in cast-iron tubing.

Regarding the sewage outfall, he asked how long an interval was allowed to elapse between putting in the concrete and the turn of the tide. His experience in close-boarding

the sides of the trenches, putting in the concrete, and then covering the boards over, had not been so successful as the lecturer's; the water got in and practically destroyed the concrete.

He agreed entirely with the President that the design of the reservoir described was such that it must fail. When there was a greater thrust on the outside it might be better to have the batter on the inside, but his opinion certainly inclined to its being on the outside wherever possible.

The action of the Local Government Board with regard to the period of repayment of loans where reinforced concrete was concerned had compelled him, in the interests of his clients, to go in for massed concrete. The Board, however, in dealing with public money, had to allow a large factor of safety, and he considered that from that standpoint their action had been quite justified so far—there was safety in going slowly. In reinforced concrete one had to deal with the human element, and unless that were carefully looked after danger would creep in.

Mr. D. B. Butler, Assoc.M.Inst.C.E., said their thanks were due to Mr. Tingle because his Paper was largely based on personal experience.

Replying to Mr. Moss-Flower's defence of the Local Government Board, his own experience was that, provided reinforced concrete was properly made, the contact of water—should it get in to a slight extent—with the lime would form an alkaline which would act as a preservative to the reinforcement.

Mr. J. G. Sharp (Messrs. Sharp, Jones and Co.) said he found the lecture very interesting, as concrete was quite the earliest material he had to deal with. In the old days, concrete was regarded as a coarse material, to be used in bulk, and not for such delicate things as drain pipes, etc.

Concerning the old tests of concrete pipes made of broken stoneware, the quantity of this latter material available at the present day was practically nil. When obtainable, it was a most excellent aggregate; but even in those far off days, with a little trouble, it was possible to obtain a good cement, although there was no demand for it.

He concurred with Mr. Moss-Flower's views respecting loans. People inquired why the Local Government Board did not follow the example of the Board of Works, the Admiralty, and other Government Departments, in regard to the use of reinforced concrete. His answer to that was, the Local Government Board had to sanction loans which would be expended under the supervision of local authorities, and the officials of local authorities were not always men of the wide experience of those in Government Departments, the result being that adequate supervision, which was most essential, was not obtained.

In conclusion, Mr. Sharp emphasised the value of finely ground cement.

Mr. S. Bylander inquired what provision was made at the joints in the reinforced concrete pipes to take up the axial force produced by the pressure of water. He had not been able to discover any method himself for keeping the two pipes together.

Mr. Sharma (Messrs. John Ellis and Sons, Ltd.) was surprised that so little had been said that evening about the importance of the aggregate in which the cement was mixed. He would like to have some expression of opinion from the lecturer as to the difference between the pot aggregate and the granite aggregate. The speaker concluded by referring to some tests he had made with granite sand, to the same specification as Leighton Buzzard sand, and said that the results obtained proved to him that even more important than the cement was the question of the aggregate, to which more attention should certainly be paid in the future.

Mr. Alan Graham, A.R.I.B.A., said that more was to be learned from failure than from success, and for that reason they ought to be grateful to Mr. Tingle for the instances of failure which he had given.

The President had asked why cruciform bars were used instead of round or square. The former were employed in order to place the value of the steel at its right position. The arm of the cross was greater in the upper bars, which performed the double work of acting as helical reinforcement and as stirrup or shear members.

Mr. Robert N. Sinclair, M.C.I., referring to the reservoir where the crack appeared on the outside of the wall, said he was interested to see the way in which the difficulty was overcome, but he thought it would have been impossible in the case of a long wall. As a rule, the crack went right through, and whatever was done in the way of cutting out and repointing would be quite useless when the temperature varied again.

On the motion of the President, a hearty vote of thanks was accorded to Mr. Tingle for his Paper.

MR. TINGLE'S REPLY.

Mr. Henry Tingle said the President had referred to a statement in the Paper that concrete should be machine mixed. By that, of course, he meant that it should be mixed in batches, and not continuously. The statement referring to hastening the setting of the con-

crete by exposure to hot air did not apply to the "Bonna" pipe, or any other pipe—it was quite general. At the same time, he would like the Institute to take the matter up, and see what was the real effect of this treatment on concrete tubes.

The President's query re the cruciform form for reinforcement had been answered by Mr. Graham.

He could not agree with Mr. Moss-Flower that cast-iron tubing would have been cheaper for the ejector chamber. They had received estimates for cast-iron tubing, but had chosen the method adopted. The same gentleman had asked how soon the tide followed the laying of the concrete. As soon as the water began to get to the level of the top boards they were closed, the water came through almost immediately, and work had to be suspended for perhaps two or three months until they had a quiet sea.

He entirely endorsed the views of Mr. Sharman concerning the great importance of the quality and grading of the aggregate.

The first reservoir referred to in the Paper was undoubtedly a failure, but the second was a failure only to the extent that, during a very hot summer, slight cracks appeared in the wall that was exposed to the sun—no water was lost, so he hardly considered it a failure. He mentioned it as showing the temperature stress where one side of the reservoir wall was heated to a temperature of perhaps 130 degrees, and the other, being in contact with water, had a temperature of only 55. Had it been a long wall it would probably have failed altogether; the method adopted prevented it from cracking right through. He agreed with Mr. Graham that the reservoir failed owing to the fact that the wall was not strong enough to withstand the pressure of the arches.

The meeting shortly afterwards terminated.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE BUILDINGS IN CHINA.

At the present time, when considerable efforts are being made to extend the field of our commercial operations, especially in China, where there are extensive openings for reinforced concrete, and where German business activities have been very considerable, it should be interesting to note that in spite of the enormous distance between England and China it has been possible to erect rapidly two large buildings in Hankow with bars which were all rolled and prepared in our country.

Several years ago two large buildings, belonging respectively to Messrs. S. W. Litvinoff and Co. and to The Trading Company, were destroyed by fire by the Imperial Chinese Army. When it became necessary to reconstruct these buildings Messrs. Hemmings and Berkley, architects and civil engineers of Hankow, gave preference



THE BURNING OF HANKOW BY THE IMPERIAL ARMY.

to reinforced concrete, and advised their clients to use this material for the reconstruction of their buildings. The Coignet system, which has been described in our pages on many occasions, was adopted; it has been extensively applied in Eastern countries on account of its simplicity in execution. Messrs. Edmond Coignet, Ltd., of Westminster, acted in the capacity of advisers concerning the use of their system, and all the bars forming the reinforcement were prepared and shipped by Messrs. The Whitehead Iron and Steel Co., Ltd., of Tredegar, Mon.



VIEW OF GODOWNS OF MESSRS. LITVINOFF AND THE TRADING CO. IN DISTANCE.

It was, of course, due to the fact that the architects had already a knowledge of the local conditions that it was possible to erect these two buildings in this material without any delay or difficulty, and entirely by means of native labour.

The difficulties which had to be overcome by the architects will be better understood when it is realised that in China work of this nature cannot be executed by local contractors, but must be carried out by the architect or engineer, practically acting also in the capacity of contractor, arrangements being made for the execution of the work with a certain number of small local contractors, each being in charge of a certain portion of the work.

All the window-sashes, the glass for same, and several other fittings were also sent from England, and the two buildings were carried out simultaneously without any trouble whatever.

It will be noticed that these two buildings, the walls of which were constructed in brickwork, have a very good appearance, which does credit to the architects; and the construction was comparatively simple. The buildings are used for the preparation and storage of tea, locally called "Tea Godowns."



THE TRADING COMPANY'S GODOWN BUILT IN REINFORCED CONCRETE.

The building for Messrs. Litvinoff and Co. measures about 300 ft. long by about 55 ft. wide, and contains a first floor and a second floor, calculated for a superload of about $1\frac{1}{2}$ cwt. There is also a flat roof over the whole extent of the building.

The building for The Trading Company is in the shape of a trapeze, having approximately the following dimensions: an average length of about 140 ft. by an average width of about 60 ft. It is composed of three floors and a flat roof.

The stairs in these buildings were also executed in reinforced concrete.



MESSRS. LIVINOFF'S GODOWN BUILT OF REINFORCED CONCRETE.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

Structural Engineer's Handbook. By Milo S. Ketchum, C.E.

McGraw-Hill Book Co. Inc., London: 6 Bouverie St. 89 pp. + xv.

Contents.—Steel Roof Trusses and Mill Buildings—Steel Office Buildings—Steel Highway Bridges—Retaining Walls—Bridge Abutments and Piers—Timber Bridges and Trestles—Steel Bins—Steel Grain Elevators—Steel Head Frames and Coal Tipplers—Steel Stand Pipes and Elevated Tanks on Towers—Structural Drafting—Estimates of Structural Steel—Engineering Materials—Structural Mechanics—The Design of Steel Details.

This volume contains data for the design and construction of steel bridges and buildings, and much time and labour must have been expended by the author in its preparation, as it contains numerous drawings and tables, each of which represents a considerable amount of work. The author himself states that the book is the result of many years' work, and it is intended as a working manual for the engineer rather than as a treatise for the use of untrained students. The various standard sections and details which are given are based on American practice, and these will possibly not be in all cases of the same value to engineers in this country as they will be to American engineers, but apart from this there is so much valuable information throughout the volume that it should find a place on the bookshelf of everyone engaged in structural design. The calculation of stresses has only been briefly considered, and then only in such cases as are not generally covered by text books, and several standard specifications for materials and workmanship are given that are not readily available to the reader from any other source.

The diagrams used throughout are very clear and satisfactory and the tables are well arranged. Tables of logarithms, junctions of angles, and tables that are easily available have not been included.

The explanations and text generally are

very clear and concise, and a practical and theoretical knowledge is displayed by the author which renders the descriptions valuable and interesting. Wherever possible, typical details are given of work actually executed, as distinct from theoretical matter prepared especially for the book, but many of the tables are original and have been calculated and arranged in a distinctive manner for this volume.

The author states that he will appreciate notices of errors in order that they may be rectified in subsequent editions, and we notice that on page 548 the radius of gyration of a rectangular section is expressed as $0.289d$, instead of $0.289d$, this being obviously a printer's error.

We have no hesitation in recommending the volume to all engineers, as we feel that it will prove useful to all those who acquire it and be a welcome addition to any library.

Concrete and Steel Construction. Part I.—Buildings. By Henry T. Eddy & C. A. P. Turner, Minneapolis. pp. 438 + xv.

Contents.—Concrete and Steel—Slabs—Beams—Beam Action and Slab Action—Theory of Flat Slabs—Computed Stresses and Deflections verified by Tests—Moments with Two-way and Four-way Flat Slabs—Reinforced Concrete Columns—Foundations—Elements of Economic Construction and Cost of Reinforced Concrete Work—Fireproof Properties—Protection of Steel—Finishings—Artistic Treatment—The Execution of Work.

This is a treatise upon the elementary principles of design and execution of reinforced concrete work in buildings, and it is illustrated throughout by numerous drawings and photographs. Although various patented and unpatented types of construction have been dealt with, the majority of the text and illustrations deal with examples of the Mushroom system, in which the authors have a particular interest. The volume is well written and arranged, and the matter is expressed in a

manner which renders it easy to understand and interesting to read. The theoretical design of the various members of a reinforced concrete structure are considered fully, commencing with those elementary principles upon which the design is based, and the formulæ recommended by the Joint Committee of the American Society of Civil Engineers on Concrete and Reinforced Concrete are given, together with the standard notation. A great deal of text is devoted to the theory as affecting the Mushroom system, and comparisons are drawn by the authors to show in what manner superiority is claimed for this system as compared with the beam and slab method of construction. There is no doubt that a large amount of time and energy has been spent by those responsible for this volume in the preparation of particulars and in investigation, in order to put forward evidence in support of their contentions, and the matter presented is both theoretical and practical in character. Several tests are described and illustrated, and many buildings actually executed in reinforced concrete are dealt with.

The chapters dealing with surface finishes and the execution of the work are very interesting, and contain a great deal of information which should prove useful.

Generally speaking, the book is quite good, and, although there is much which would only prove useful to anyone adopting the Mushroom system, there is also a great deal of matter which is worth while studying, and the authors have prepared the volume in a thorough manner which is commendable.

Concrete Roads and Kerbs.

Published by the Associated Portland Cement Manufacturers (1900) Ltd., Portland House, Lloyds Avenue, E.C.

The question of concrete roads is receiving considerable attention at the moment, and therefore this pamphlet will doubtless be read with interest by those who are specially studying this important matter. It is illustrated, and contains extracts from papers and reports presented at the International Road Conference held in London, and the Concrete Road Building Conference held in Chicago. There is further reference to the experimental road sections now in course of construction at Gravesend, as also the road recently completed at Chester. A chapter is also devoted to concrete kerbs. The pamphlet should appeal specially to engineers and surveyors, who can obtain copies on application to the Publications Department of the above company.

CORRESPONDENCE.

Under this heading we invite correspondence.

"The Use of a Tremie for Underwater Concreting."

SIR,—Your last issue contained an article on the use of a tremie for underwater concreting, and I thought that my own experience during last summer with a tremie might prove interesting.

The reinforced concrete abutment bottom slab of a concrete bridge across the Tweed, near Peebles, had to rest upon a 3-ft. thick mass concrete foundation, the bottom of which was 5 ft. below water surface, and for depositing this the use of a tremie proved most satisfactory.

A wooden tremie with parallel sides was first tried, but soon discarded, because the concrete, which was only just damp, arched across and made the combined weight of concrete and wetted wood too much for two men (more could not be employed because of space) to lift.

The local blacksmith was called upon to make a galvanised iron tremie with hopper head and fitted with handles, and it worked splendidly.

Tremie.—The length was about 11 ft., including the conical hopper head, which was 30 in. across at the top, 12 in. high, and 6 in. where it joined the pipe, which was of that diameter at the top, but increased to 10 in. in diameter at the bottom to prevent the concrete arching, and which therefore allowed easy lifting. Two handles, one on each side at the mid-height, allowed one man to lift, while another handle near the bottom allowed another man to guide the tremie and so spread the concrete. During the placing of the first layers a rope was run through this bottom handle, but as the concrete rose the man caught the handle by hand (it was placed 1 ft. from the bottom of pipe). The art is in maintaining a level surface to the concrete, thereby preventing rolling and loss of cement.

The Concrete.—The concrete was mixed 1 : $2\frac{1}{2}$: 5 of gravel, and on pumping out the water it was found to be quite dense, and over an area of 27 ft. by 16 ft. an occasional turn with a diaphragm hand-pump easily kept the surface free of water, a slight pitch to a sump in one corner making this easier. The concrete was, as previously noted, mixed only just damp, and I would recommend that as little water as possible, consistent with easy movement in the tremie, be used for such work.

The tremie was always kept half full, and only lifted about two or three inches for each half pailful of concrete added.

In slowly moving water a trumpet mouth to the lower part of tremie will greatly protect the concrete from action by water.

H. C. JOHNSON, University College, Cork.

POPULAR USES.

Under this heading it is proposed from time to time to present particulars of the more popular uses to which concrete and reinforced concrete can be put, as, for instance, in the construction of houses, cottages, and farm buildings. Previous articles will be found in our issues of December, 1912; January, March, July, October and November, 1914; and January of this year.—ED.

SOME CONCRETE TROUGHS.

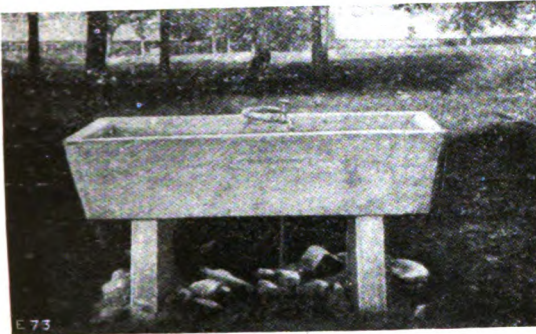


FIG. 1. A CONCRETE FIELD TROUGH.

The most common practice is to build circular troughs of monolithic concrete, but blocks also may be used. One of our illustrations shows an excellent barnyard trough made of silo blocks, laid up three courses above the foundation and plastered on the inside with rich cement mortar about 1 in. thick at the top and 2½ ins. thick at the floor level. Reinforcing rods were laid in the joints between block courses, and the trough has stood several winters without sign of injury. Plenty of reinforcing, a battered or sloped inside wall, and a rich concrete make a trough that will withstand frost.

Rectangular tanks are a little easier to build, especially if the farmer builds his

In America, on the farms, concrete troughs are more or less taking the place of the wooden trough, which has a comparatively short life and soon becomes leaky and dilapidated in appearance.

There are many shapes and kinds of watering troughs. The most attractive in appearance is the circular trough, but this is a little harder to build than one of rectangular shape. The circular tank shown on this page was built by farm labour, and is filled from a well by a pump operated by a windmill.

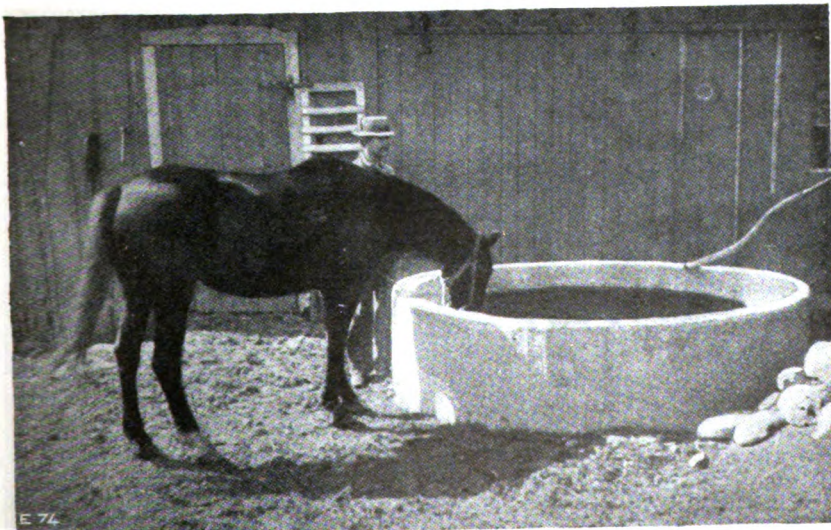


FIG. 2. A CIRCULAR TROUGH.



FIG. 3. A CONCRETE MONOLITHIC FIELD TROUGH.

smaller for horses and the larger for stock. The windmill pumps water directly into the larger tank, from which an overflow pipe fills the horse trough.

Around the latter is a concrete apron to prevent the formation of a mud-hole. This is a very good practice, for otherwise the animals' hoofs dig up the ground around the tank and form holes in which water will stand.

The horse trough was built later than the one for stock, and the illustration (Fig. 6) shows it to have much smoother walls. It is possible to make perfect concrete work the first time, but generally the surfaces are cleaner and the corners more perfect after the builder has had a little experience.

A still better form of covering, and one that is specially suited to large tanks, is shown in Fig. 7. The tank is 10 ft. by 30 ft. by 6 ft. deep. An arched concrete cover keeps out dirt and cold, and two wooden covers at

the end protect the openings where stock is watered. To build the walls is a simple matter. The arched top is more difficult to construct, although one who has had a little experience with concrete need have no fear to attempt it. The concrete cover was built in three sections, 10 ft. at a time, and the forms used for the first section moved on



FIG. 4. A RECTANGULAR STOCK TANK.

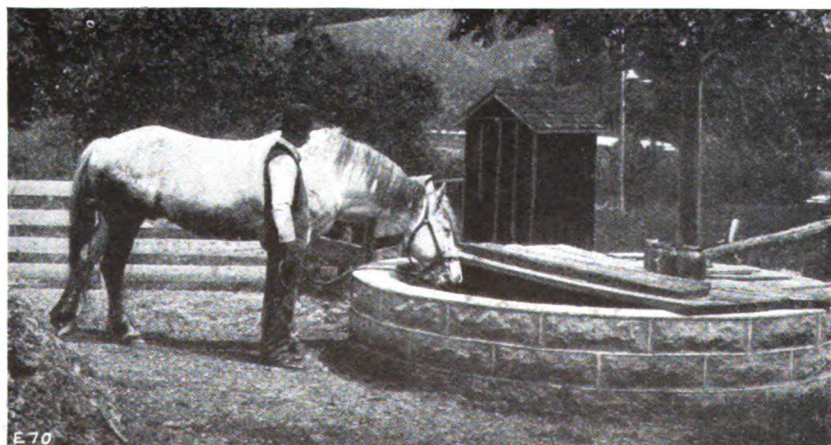


FIG. 5. A DRINKING TROUGH OF CONCRETE SILO BLOCKS, PLASTERED ON THE INSIDE.

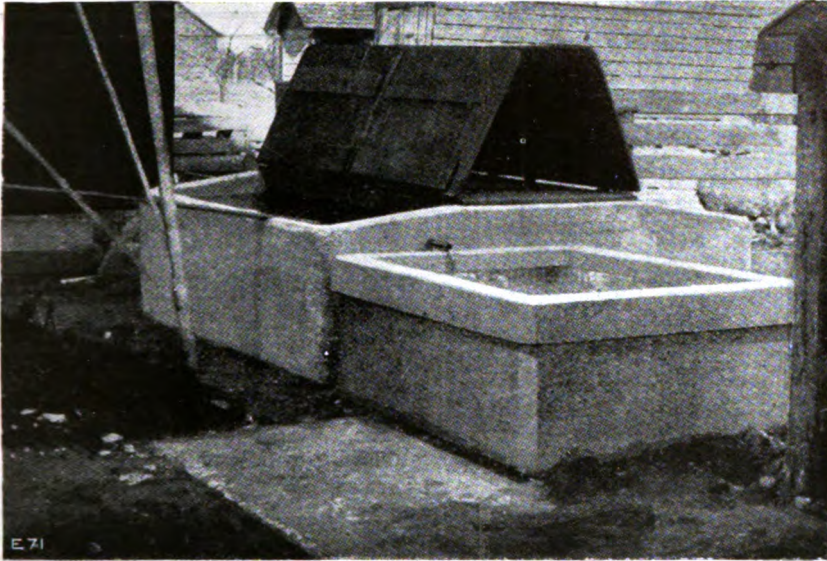


FIG. 6. SHOWING COVERS FOR CONCRETE TROUGHS.

after it had hardened. Thus only a small amount of lumber was necessary. Where a large storage tank is needed there is nothing better than this type.

Often there is need of a smaller trough to be placed in a field temporarily. This also can be built of concrete to advantage, as shown in *Fig. 1*. They are 5 ft. long, 2 ft. wide, 15 ins. deep, well reinforced, and stand on two concrete supports. The trough is piped to a pressure supply.

A clean-walled water-tight drinking trough for stock is a source of permanent satisfaction to the owner. There is no calking of joints, for the concrete trough has no joints. There is no trouble from freezing, for a well-designed tank can freeze solid without harm.

The above particulars and illustrations have been taken from the *Farm Cement News*, U.S.A.

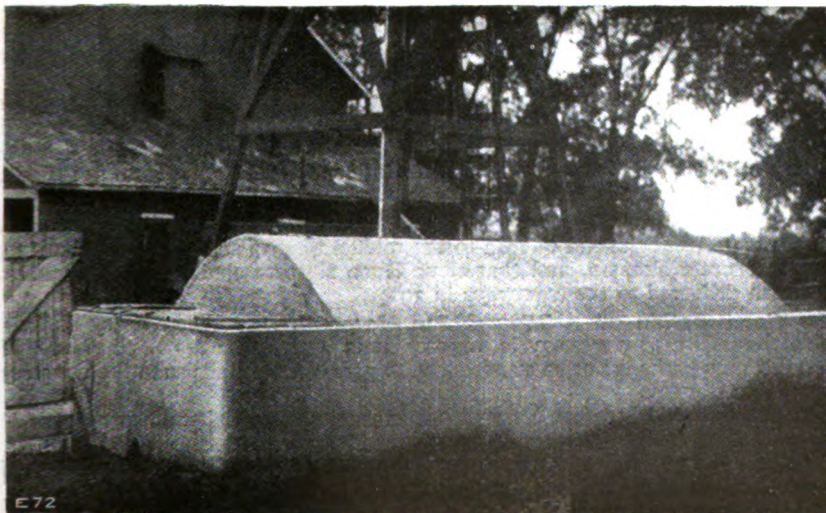
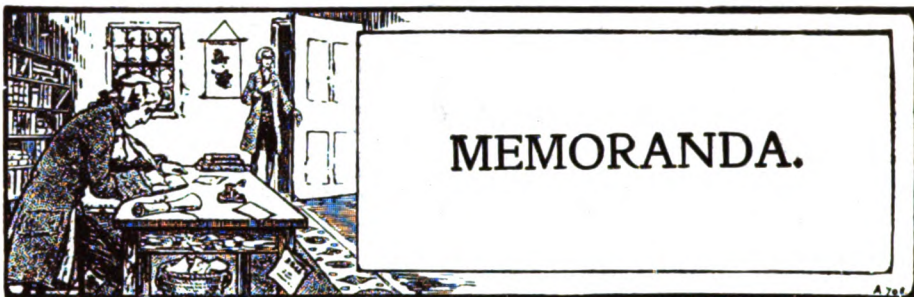


FIG. 7. COVERED CONCRETE WATER SUPPLY TANK AND TROUGHS.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Bristol Docks.—A tender has recently been accepted for the construction of a reinforced concrete wharf at the oil berth in the Western Arm of the Royal Edward Dock, Bristol. The tender was that of Messrs. W. Cowlin and Son.

Belfast Harbour Equipment.—The new 120-ton crane which has just been brought into use at Belfast Harbour rests upon three concrete piers at an elevation of 47 ft. above quay level. The foundation of the main pier (which measures 40 ft. by 40 ft. at its base) has been carried up from a depth of 52 ft. below quay level, and contains 4,000 tons of concrete. This portion of the work was carried out departmentally by the Commissioners' employees. The total distance from the foundation level to the top of the jib of the crane when at its minimum radius is 204 ft.—about 20 ft. greater than the height of the larger of the two chimneys at the new Graving Dock.

Alford.—At a recent meeting of the Alford Urban District Committee it was agreed to recommend the County Road Board to erect a concrete bridge with a 13-ft. carriage-way over the Don at Drumallochie, the estimated cost being £1,575.

New Reinforced Concrete Bridges.—At a meeting of the Thames Conservancy last month provisional acquiescence was expressed with the plans for three new bridges across the Thames. One of these would be constructed across the river between Goring and Streatley. Plans were originally prepared two years ago, but had to be abandoned because it was found to be impracticable to construct such a bridge with approaches having a reasonable gradient to suit the structure. Consequently another plan has been prepared, providing for a tresselled structure somewhat similar to the present bridge, but to be constructed in reinforced concrete. This design had been made the basis of the plan which had now been deposited. The Conservancy suggested a modification in the design, with the object of increasing the width of the waterway for navigation purposes.

The other two bridges were to be built by the Reading Corporation over the river between Reading and Caversham. The first to be built would be the De Bohun Road Bridge, which would be of reinforced concrete with a stone parapet. The design provided for a bridge of single span with a width of 180 ft., and a maximum rise of 18 ft. above the normal level of the water.

Concrete Tramway Sleepers.—We understand that the newer lines of the Berlin Tramways were from the end of 1912 to the middle of 1913 supplied with about 25,000 reinforced concrete sleepers, and that some 8,000 sleepers similarly constructed were to have been delivered to the tramways in 1914.

The Construction of a Slipway for Motor Boats.—Where a dry dock is not available the utility of a slipway is unquestionable, says *The Motor Ship and Motor Boat*, and it may perhaps not be out of place to give a few details as to the construction of a workable slipway.

The slip described here has been with a few unimportant alterations constructed in India, and has worked satisfactorily for several years.

When building a slipway three important factors have to be taken into consideration—tide, ground available, and depth of water at the lowest portion of the slip. The gradient can always be formed.

In selecting the site it is best where possible to pitch on a position where the river shoals rapidly to about one fathom at low tide. Having selected the site, the question of tide must be gone into, and for the sake of argument we will assume that the locality chosen is somewhere on the lower reaches of the Thames, where the tidal difference is 18½ ft. at spring tides and 15 ft. at the neaps. Now the slip must be capable of taking on a vessel of at least 60 ft. in length and 6 ft. draught forward, so that the lower part of the gradient must, at, say, three-quarter flood, have a sounding of 6 ft. plus the height of the slip carriage, and the higher part of the slip must terminate at least 65 ft. above high-water mark. The extra 5 ft. is required for drawing tailshafts, etc. As the gradient for convenient working should not be more than 1 in 15, it will be seen that the total length, allowing 6 ft. clearance at the upper end for the winch gear, is 360 ft. These figures can, of course, be modified by only docking at full flood and working when the ebb has well set in.

The length having been decided upon, a post should be driven in at high-water mark, with its end just level with the water, and another 71 ft. above it, the top being 4 ft. 8 4-5 in. above the level of the high-water post.

These two posts are standards for gradient besides being checking points for length. Another post is driven in at the river end 280 ft. below the high-water post, and the high-water level should be marked on this.

This post indicates the lower point of the gradient, and at very low tide should mark the edge of the water. If the soil is fairly stiff, such as good London clay, it need only be removed for a distance of 2 ft. to 2 ft. 6 in. below the gradient level, and the space filled in with concrete. This requires to be of very good quality immediately below the rails, but need only be about a foot thick in between and at the sides. The width of the cutting should be about 20 ft. at the bottom and about 30 ft. at the top. This depends, of course, on the earth, as, for instance, should the soil be of a loose, sandy nature, the angle of slope must be much less than if of stiff or hard clay.

The rails, which are preferably of the double-headed variety, should be of not less than 60 lb. weight and of the bull-head pattern. Old rails in good condition are quite serviceable for this class of work. The centres should not be less than 6 ft., and the best method of attaching them to the track is to have 12-in. oak baulks running longitudinally for the whole length of the track and flush with the concrete surface. The chairs can be attached to these, and the rails keyed in, in the usual manner. The outside edges of the baulks should be about 6 ft. 2 in. apart, and this will give quite room enough for the ordinary 14½-in. chair to be bolted on to them. On the inside of these baulks there should be another pair of baulks, 4 in. by 4 in., to carry the rack rails. These are to be not less than 4 in. wide, and should be bolted through by bolts in countersunk holes to the concreted-in baulks. To prevent any possibility of the baulks themselves slipping, anchor-spikes or Lewis bolts should be employed, these being embedded into the concrete for about 12 in.

The 71 ft. of working section should have the banks well dressed and three flights of concrete steps on each side, about 2 ft. 6 in. broad. These are very useful to take the ends of struts and also to rest the shore ends of gang planks on when material is being taken on board or brought off. With the centre 6 ft. from the top end two recesses should be constructed in the bank about 10 ft. long and 6 ft. deep. These are to accommodate a pair of sheer legs for use in removing engines, etc., from the boat, and the sides, which may be made vertical, can be bricked up. A nice finish to the whole construction is to brick or concrete the slope up for about 6 ft. and then dress the top down with coir to a depth of 6 in., finishing the surface off with expanded metal and cement 1 in. thick. This makes a very clean job, and the whole can be hoised down, ensuring freedom from dust when varnishing or french-polishing is being performed.

Draining Land by Wells; Concrete Drain Head.—Draining agricultural and swamp lands by wells or vertical drains—instead of by ditches and horizontal lines of tile—has been done successfully in a number of cases throughout the Central States. The wells are sunk to reach underflow strata that will absorb or carry off water readily. Success depends largely upon proper knowledge of the geological conditions, and results are problematical when the drain-wells are sunk at random.

The depth of wells is said to range from 20 ft. to 45 ft. It is claimed that one well will drain from half to three-quarters of an acre.

The wells are drilled with augers, pipe extensions being added to the shaft of the auger as the depth increases. Through quicksand or soft clay, casing must be put in. A bucket to remove soft materials and a stone-picker to remove stones or small boulders will also be required. The nature of the materials encountered must be observed carefully. The hole can be inspected by means of a mirror to ascertain the strata reached, etc. The diameter of the hole is usually 8 in., to take a lining of 6-in. drain tile.

The drainage is taken from the soil, surface water being excluded. For this purpose special drain heads are made, which prevent the water from carrying soil into the pipe. The top of the drain pipe is 3 ft. or 4 ft. below the surface of the ground, and above it the ground is excavated to a diameter of about 3½ ft., the lowest part of this hole being lined with stones or old brick to form a water chamber. Upon the 6-in. drain pipe is placed a collar, and this supports a grooved inner head surrounded by a sleeve or soil fender. The chamber is roofed with a cap slab. After these parts have been assembled, the ground is filled in above the cap. When the ground becomes wet, the water seeps up under the chamber and siphons up under the fender, flowing over the top of the grooved inner head into the vertical drain.—*Engineering News*.

PUBLICATION RECEIVED.

The British Engineers' Association.—In our last issue (page 57) we made a short reference to Mr. Wilfred Stokes's speech regarding our foreign trade and the war. The above Association has now published the entire speech in pamphlet form, together with an article from the *Engineer*. The pamphlet is obtainable at 32, Victoria Street, S.W., price 6d.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum ¼-in. Steel Plate.

The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

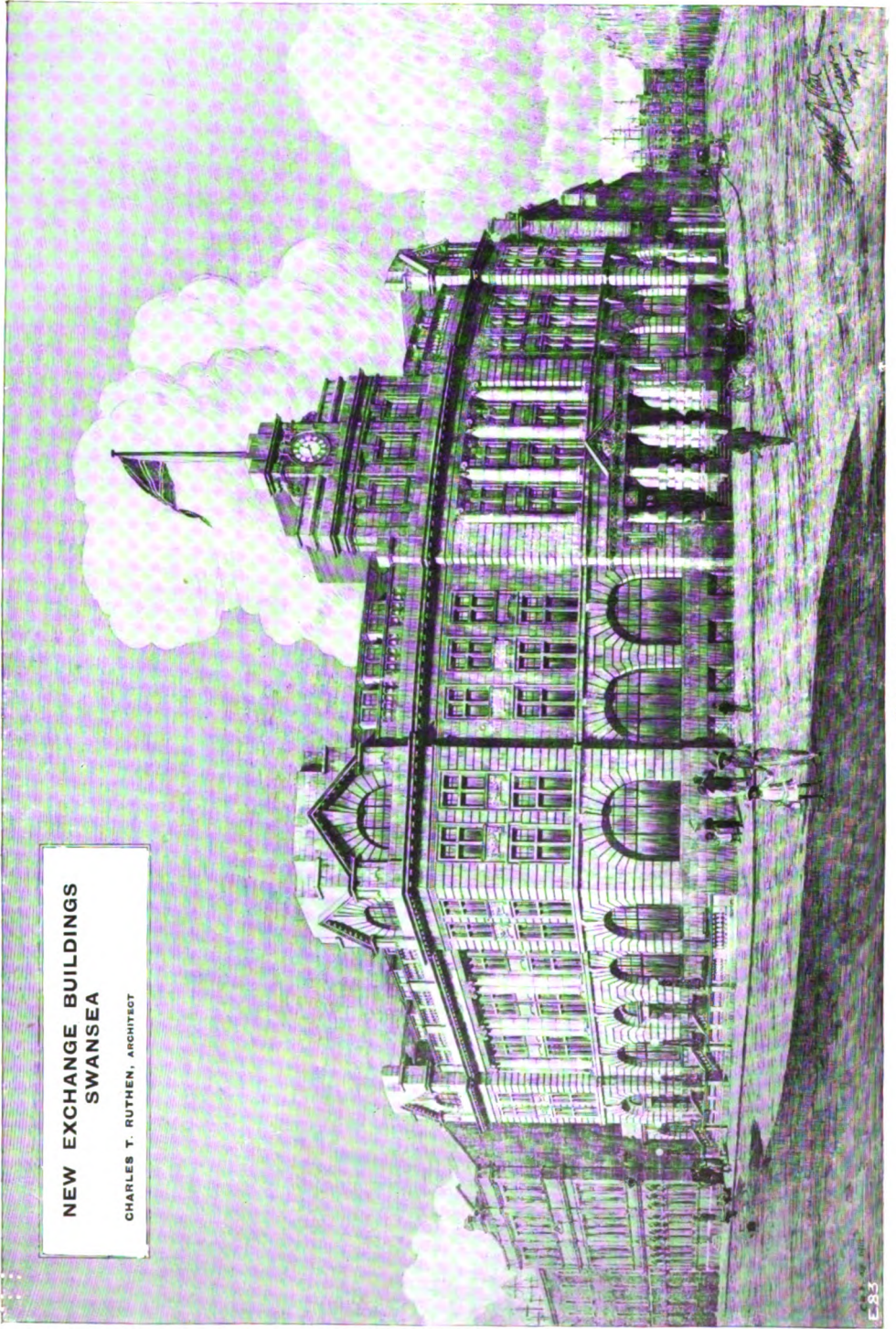
is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.

NEW EXCHANGE BUILDINGS
SWANSEA

CHARLES T. RUTHEN, ARCHITECT



Exterior View.
New Exchange Buildings, Swansea.
(For description see page 117.)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X. No. 3.

LONDON, MARCH, 1915.

EDITORIAL NOTES.

RURAL HOUSING.

THE problem of erecting cottages at a reasonable cost for the occupation of the working classes is one which has received a great deal of attention during recent years, and it is a question of much importance to a large section of the community. The designer and contractor are affected during the ordinary course of their business; the financier is concerned, inasmuch that he requires to see a satisfactory return on the initial outlay in any building scheme, and the working classes must be able to acquire dwellings the rent of which is within the limit they can afford to devote out of their wages. Many ideas have been put forward, and various materials and methods have been suggested and in some cases adopted, in the attempt to provide a solution of the problem; but it is still extremely difficult to erect satisfactory cottages at a cheap rate. In many districts the bye-laws which govern the methods of building are so rigorous and unreasonable that they hamper the development of schemes which would otherwise go forward; and although we do not advocate the abolition of a proper control of all classes of building work, we do consider that a certain amount of judgment should be exercised and each case dealt with according to circumstances, and generally speaking the bye-laws in existence are antiquated and not applicable to modern work.

At various times we have pointed out the possibilities of concrete for cottage work, and although it has not yet been tried on a very extensive scale, we are confident that it has a great future in this class of construction. In this issue we give details and description of some cottages which are being erected at Crayford, in Kent, and in this case concrete blocks have proved cheaper than brickwork, despite the fact that Crayford is a brick district and a brickfield actually adjoins the site; thus there is no question of bricks being unobtainable or even procurable only at a high price. If concrete can compete favourably with brickwork in a district such as this, it will be clear that a tremendous advantage will be shown in the majority of cases where cartage is an important factor in the cost. The work has been carried out during the worst months of the year for building, and yet it has been executed quickly and with satisfaction to all the parties concerned. The cheap price per cottage is all the more remarkable when it is considered in conjunction with the increased cost of materials,

and as materials are likely to still further rise in price, concrete should become more general in use, as in this form of construction there is only about one-half the cost of labour in laying the blocks as compared with brickwork, and less mortar is required.

The strength of concrete and its lasting properties are unquestionable, and it only remains for contractors to see that work of this kind is executed in a proper manner, and any prejudice would be removed from the minds of building owners, and its cheapness would lead to the adoption of concrete block construction for many housing schemes.

THE CONSTRUCTION AND PROTECTION OF BUILDINGS IN RELATION TO FIRE.

THE Society of Architects were fortunate in the presentation of a most useful Paper by Mr. Alban Scott, their vice-president, whose subject was the Construction of Buildings in Relation to Fire.

In another portion of the current number we reproduce an abstract from this Paper, which should be perused with care, as being eminently instructive and also valuable for reference purposes.

It is obvious that in the development of fire protective construction concrete has already played a great rôle, and this rôle is being increased from year to year. The excellence of concrete as a fire resistant, if proper aggregate be used and if properly applied, scarcely requires argument. But there is still an extraordinary amount of ignorance as to the nature of the aggregate, as to its size, and as to the thickness to which the material should be applied in order to be thoroughly efficient.

One thing, however, is quite plain in the negative sense, and that is, that there is no concrete of such small fire resistance as concrete of Thames ballast or similar ballast, especially if the aggregate does not pass through a $\frac{3}{4}$ -in. mesh and if it contains flints.

Coke breeze aggregate is the most valuable fire resistant of all, but unfortunately the purveyors cannot be sufficiently relied upon at all times to deliver it in a properly cleansed and washed condition, and if not properly cleaned it may contain chemicals that affect metal work. This has probably resulted in the somewhat short-sighted policy of the London County Council, and some of the institutions it has consulted, in desiring that coke breeze concrete should be avoided.

What would of course be ideal, and what is now becoming common in the best American practice is, for the reinforced concrete work to be carried out in some one of the stronger aggregates, and that this reinforced concrete work should, particularly where used for column work, be protected by $1\frac{1}{2}$ in. or 2 in. of coke breeze or cinder concrete, which afford such an excellent protective covering.

The whole subject matter is, however, one that goes too far to allow of its treatment in this place, but we are afraid that an extraordinary amount of amateurism still exists as to the pros and cons of the various concrete aggregates from the fire point of view, based on opinion as distinct from facts, and until facts are given precedence to opinions, a large number of blunders will still be made.



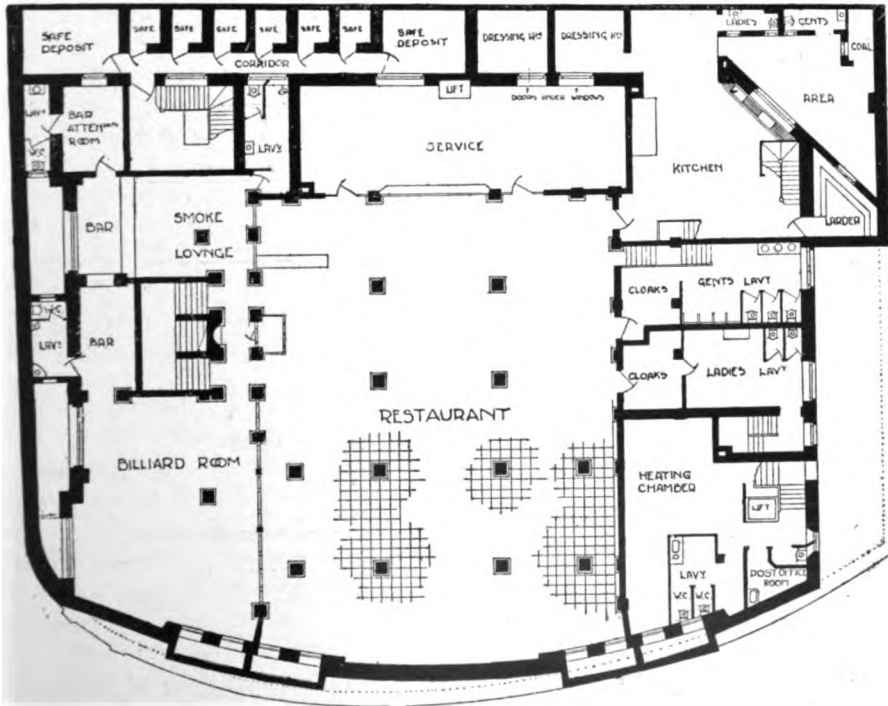
Finished Building, Exterior View.
NEW EXCHANGE BUILDINGS, SWANSEA.

THE NEW EXCHANGE BUILDINGS, SWANSEA.

The buildings here illustrated afford an interesting example of the architectural possibilities of reinforced concrete.—ED.

THESE important premises, which were formally opened in January by the Right Hon. Sir Alfred Mond, Bart., M.P., have been erected at a cost of about £30,000 upon the same site as the old Exchange Buildings stood.

The old buildings, which formed the premises wherein met the men who transact the great bulk of the business of the important docks of Swansea,

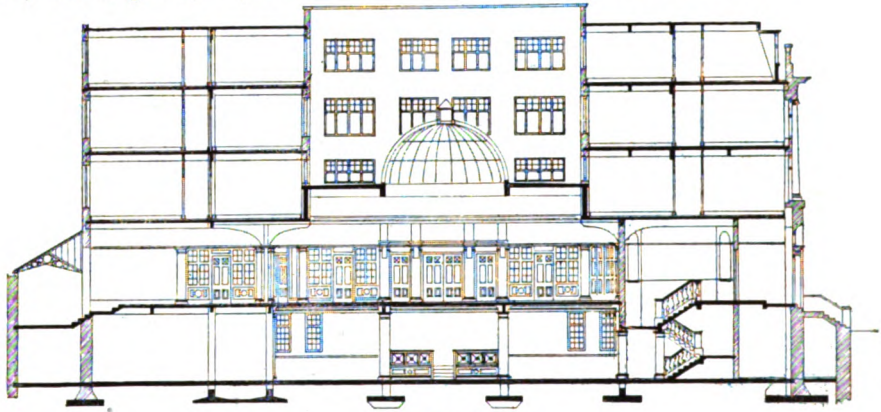


• LOWER GROUND FLOOR PLAN •

Fig. 1.

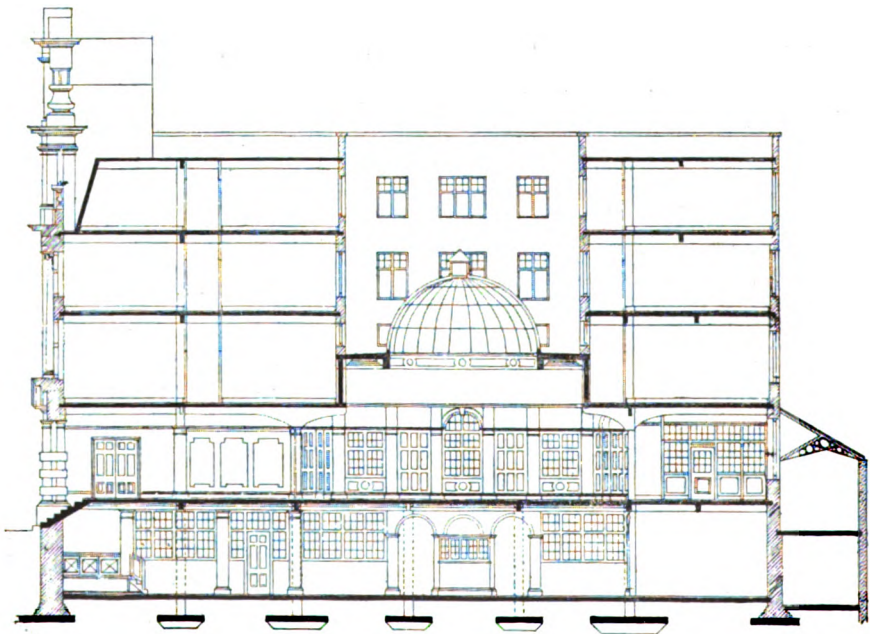
NEW EXCHANGE BUILDINGS, SWANSEA.

were in an exceedingly dilapidated condition, having generations ago been the Old Burrows Chapel, one of the chapels associated with the Countess of Huntingdon's Connexion. This site, on the corner of Adelaide Street and Cambrian Place, was originally a portion of the foreshore washed by the tide.



SECTION A-A

Fig. 2.



SECTION B-B

Fig. 3.

NEW EXCHANGE BUILDINGS, SWANSEA.

After the removal of the old premises and the carrying out of the excavations for the basement premises, it was found that the area of the site consisted largely of sand with a somewhat peculiar formation of clay "pockets" here and there.

The adoption of reinforced concrete was considered to be necessary in order to ensure a firm and solid structure, and the drawings show in general outline the arrangement of columns, concrete rafts and beams.

The whole of the foundations, stanchions, floors, beams, and lintels have been carried out in reinforced concrete, and the peculiar necessities of the accommodation required called for special treatment in reinforced concrete construction for the purpose of carrying a portion of the three upper floors over the central well area.

This design, it will be noticed, provided the large Chamber of Commerce Hall on the upper ground floor, but at the same time permitted the planning of a double row of offices on three sides of the central area.



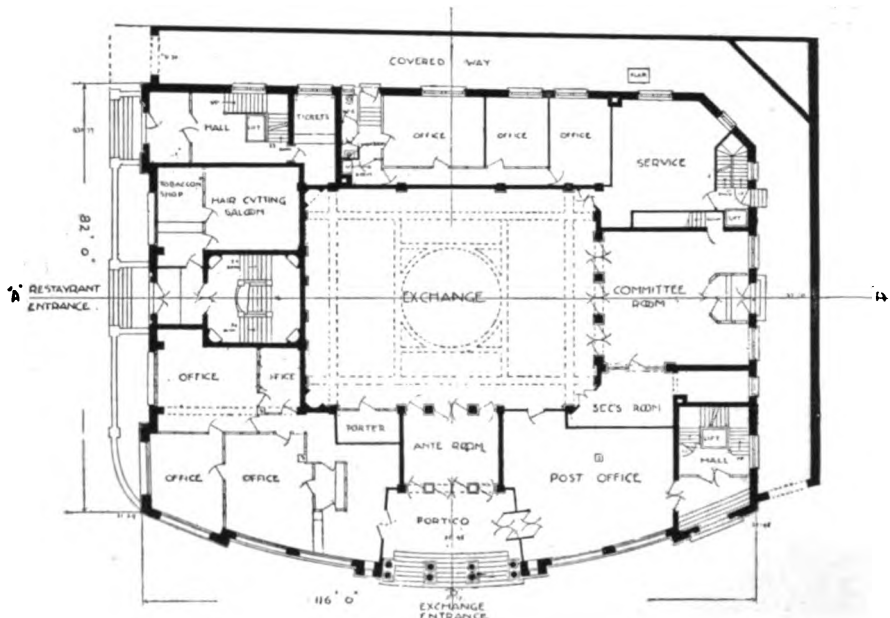
Fig. 4. The Exchange Room.
NEW EXCHANGE BUILDINGS, SWANSEA.

The lower ground floor has been planned as extensive restaurant premises, the large dining-room being capable of accommodating about 600 persons. The floor is laid in Belgian marble tiles, and the reinforced concrete columns have been artistically treated by casing in beautiful figured Italian marble.

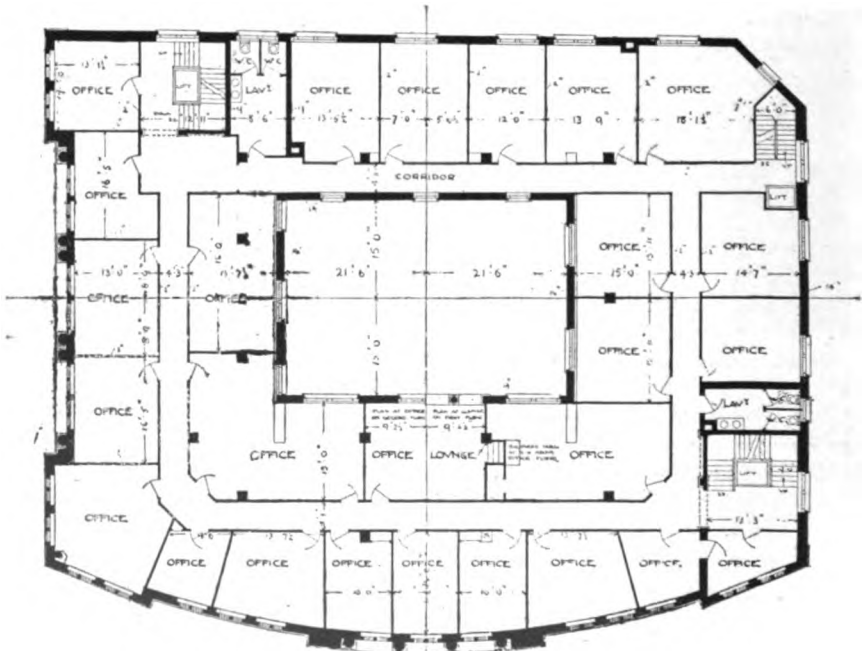
The upper ground floor contains the Chamber of Commerce premises, the reinforced concrete cantilevers being artistically treated in fibrous plaster decoration, and the deep beams above being panelled; there is also a post office, bank and office premises.

The central hall for the use of members of the Exchange is centre lighted by means of a fine stained glass dome.

The first, second and third floors contain some eighty offices.



• UPPER GROUND FLOOR PLAN •
Fig. 5.



The main façades have been erected in unpolished Aberdeen grey granite to the plinth level and above in selected Portland stone.

The balustrading in Adelaide Street has also been erected in grey unpolished granite.

Two press-button automatic passenger lifts are provided for service to the various floors, and other electric lifts for restaurant service, etc.

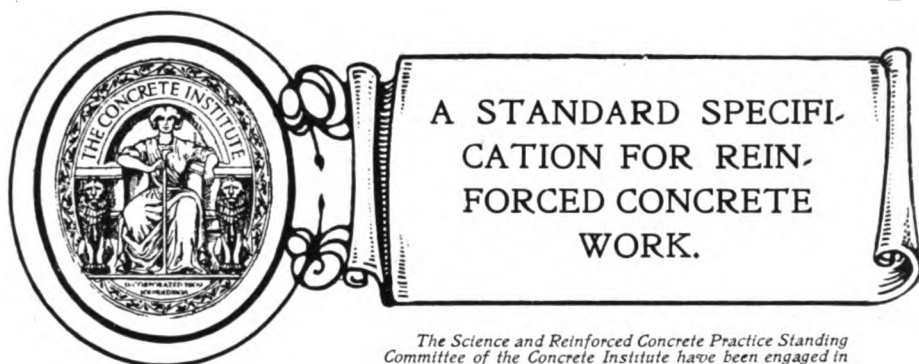
The architect for the building was Mr. Charles T. Ruthen, Licentiate R.I.B.A., M.S.A., Swansea (to whom we are indebted for our particulars), in association with Mr. E. G. Allen, F.R.I.B.A.

The reinforced concrete work was carried out by the Midland Counties Reinforced Concrete Co., of Birmingham, whilst the general contractors were Messrs. Henry Billings and Son, Swansea.

Amongst the sub-contractors were Messrs. Henry Hope and Sons, Ltd., Birmingham, who carried out the heating work; Messrs. Furneaux and Thomas, Swansea, were responsible for the electric lighting; the lifts were those of Messrs. Smith, Major, and Stevens, Ltd., Northampton, and the ironwork that of Messrs. Hart, Son and Peard, Ltd., London.



Fig. 7. Restaurant.
NEW EXCHANGE BUILDINGS, SWANSEA.



The Science and Reinforced Concrete Practice Standing Committee of the Concrete Institute have been engaged in the preparation of a Standard Specification for Reinforced Concrete Work, and the results of their labours were submitted to the members of the Institute in the form of a Draft Report at the Fifty-Sixth Ordinary General Meeting on Thursday, February 4th, in order that they might have an opportunity of making suggestions and discussing the subject generally, previous to its issue in final form. We have been asked not to publish the specification at present, but in the following article we give a few notes regarding same, together with one or two suggestions and a short account of the discussion which followed.—ED.

THERE is undoubtedly a great need for a standard specification, and the Committees concerned in its preparation are to be congratulated for taking the matter in hand and producing what, in our opinion, is a very excellent result. It would be quite impossible to put forward as a standard a specification which would be accepted by everyone concerned as ideal, because the interests of the different parties are so varied, and what would be considered as necessary by the architect would possibly be deemed excessive by the specialist. The great object to be kept in view is that the specification must be adequate and at the same time it must be fair to the contractor. If it is not adequate it will not ensure the client's interests being protected, and therefore it would not be generally adopted by engineers and architects; while if the conditions are irksome to the contractor, it will tend to increase the cost of the work without necessarily improving the class of work obtained.

It is worthy of note that in presenting the Report Mr. E. Fiander Etchells, F.Phys.Soc., stated that on almost every single point of the specification most of the members of the committee differed, but they had endeavoured to make it a consistent whole, and to embody the ideas current among all engineers as good practice. This variation of opinion is unavoidable, and as the report represents a consensus of opinion, and is based upon good practice, the Institute can have every confidence with the specification as drawn up, and feel that a great benefit will be conferred on the engineering profession and contractors generally by the issue of such a specification. It is divided into two main parts, the first of which deals with the General Provisions and the second with Materials and Workmanship.

GENERAL PROVISIONS IN THE SPECIFICATION.

With regard to the former the items are few and of a general nature, as the general conditions of contract are specified to be those of the Royal Institute of British Architects, which is a satisfactory method, and there is only one item which is, in our opinion, questionable.

This item is No. 4, which refers to the *Infringement of Patents*, and herein the contractor is called upon to indemnify the employer against any action or loss sustained owing to any infringement of patent rights. This would be quite satisfactory in cases when the scheme was designed by a specialist firm who also executed the work—a case which is the exception rather than the rule—but when the scheme is prepared and the drawings supplied by a consulting engineer the contractor can hardly be expected to control the question of infringement of patents, which is rather a point for the consulting engineer, who should take the responsibility, and there should be no risk of any infringement taking place if the engineer understood that the

responsibility was on his shoulders, as it is not necessary to adopt a patented system the rights of which belong to another party.

MATERIALS AND WORKMANSHIP.

In the second part the materials are dealt with under the varied sub-headings, such as cement, sand, etc.

Cement.—The cement is specified to conform in every respect with the British standard specification for Portland cement, and the contractor is called upon to supply a signed certificate from the manufacturer with each consignment to show compliance with the standard specified.

Provision is also made for *testing* by an independent testing engineer after delivery on the works, and when required tests of the crushing strength are also to be made in addition to the neat tensile tests. The cost of the testing is covered by an item in the general conditions, which calls upon the contractor to provide an amount based upon a certain percentage of the contract sum, this sum to be deducted in full or expended as directed; and this is undoubtedly a satisfactory method.

The *storage of cement* is covered by clause 18, which provides that the cement may be used direct from the bag if used within 14 days of delivery. If kept longer than this period it must be stored in air-tight bins, and in any case the bags must be kept in a perfectly waterproof and air-tight shed.

It will need a great deal of care to see that this clause is rigidly adhered to by the contractor, but if this is done it will prove a good clause; as the storage of the cement is far too often not considered a sufficiently important matter by those executing the work.

Sand.—Three clauses are devoted to the nature and size of the sand to be used, and these cover all the requirements for good work.

Coarse Material.—This term is used to denote all ingredients of the concrete except the cement and sand, this being considered preferable to the word “aggregate,” which is somewhat uncertain in its meaning, and the clauses relating to this are important. The nature, sizes, cleanliness and porosity are dealt with, and very wisely a list of materials which are not to be used is given, and by it coal residues and slags are prohibited. *The materials not permitted are as follows:—*

- (a) Coal residues, including clinkers, cinders, ashes, coke breeze, pan breeze, slag, or similar material.
- (b) Blast furnace slag, copper slag, forge breeze, dross, and other similar materials.
- (c) Sulphate, including plaster of paris and other similar materials.
- (d) Broken bricks containing dust, soot and mortar refuse, or bricks from clay containing free lime or pyrites.
- (e) Limestone, magnesian limestone, marbles and other calcium carbonates, if the concrete is required to be fire-resisting.

There are some materials in that list, such as coke breeze, which has been used in the past, and often with disastrous results, and we are glad to see that they are definitely barred and good work ensured as far as possible. One clause deals with the *cleanliness of the water* to be used in mixing; and the proportions, mixing and consistency of the concrete are next dealt with. The *proportions* given are for four different classes of mixture, and they vary from a 1 : 2 : 4 to a 2 : 2 : 4 mixture, and the cement must be proportioned by weight.

Mixing.—The method to be adopted when hand-mixing is employed is fully described, and the clauses relating to the concrete and mixing appear to fully cover the requirements of good class work.

During the discussion a member suggested that it is preferable to mix the sand

and cement together, instead of mixing the cement, sand and coarse material during the first turn, and there is a good deal to recommend his suggestion, which has been adopted by a large number of engineers in the past.

Reinforcement.—The clauses relating to the reinforcement appear to cover all the necessary items required in a complete specification, and we are glad to see that the adoption of higher carbon steels is allowed for, as there is a great possibility of their use, which is advocated by some engineers; and in spite of the controversy which exists on the point, and the fact that the Engineering Standards Committee are not yet prepared to advise their use, any specification which is put forward as a standard at the present time is not complete if it precludes their use.

The various properties and tests are specified, and all the steel is specified to be stored under cover; and anchorage, cleanliness, welding and bends are dealt with.

Placing of the Reinforcement.—A detailed clause is given under this heading, and this is important, as we have seen many instances where the work has been done badly in this respect, not so much on account of negligence possibly, but due to the contractor not fully realising that the whole value of a member may be upset by the faulty disposition of the reinforcement.

Provision is made for binding the bars together where necessary, and for keeping the reinforcements at the proper distances from the forms.

Forms.—The clauses relating to the reinforcement are followed by those dealing with the forms, and the first of these might, we think, be slightly amended with advantage. It is stated that the forms shall be approved by the engineer or architect, but the contractor shall be responsible for their sufficiency. It seems rather curious to invest the responsible person with the power to approve, while someone else accepts the responsibility. It would be preferable to state that the forms shall be *inspected* by the engineer or architect, and strengthened or improved in such manner as he shall direct if so required, but such inspection shall not relieve the contractor of his responsibility as to their sufficiency. We offer this suggestion to the Committee for their consideration, as we feel that the clause as at present drawn up does not define the positions of the respective parties in a satisfactory manner.

The remaining clauses dealing with the construction of the forms, their nature and removal, are sufficiently complete to cover all the necessary requirements of good work, and seem to be well expressed.

Concreting.—The methods to be adopted in the process of concreting are covered by no less than thirty-four clauses, and, although several points of minor importance were raised by members during the discussion, these clauses can be considered as satisfactory. The placing of the concrete expeditiously, breaks and stoppages, and restarting against concrete already hardened, which are important matters, are dealt with in such a complete manner that they leave nothing to be desired. Defective work, the protection of the concrete, and the suspension during frosty weather, are dealt with, and it is specified that all concreting shall be entirely suspended when the temperature is less than 35° Fahr.

Surface finishes are dealt with, and the granite finish for floors, staircases and landings is specified in detail.

Piles.—Special clauses are introduced with regard to piles, these relating to the removal of the forms, the time of driving, and the lifting or rolling after maturing.

Fittings, etc.—Five clauses are given under this heading, and these deal with the question of the fixing and position of pipes and ducts, and they are necessarily somewhat general in character, as each scheme would require to be dealt with on its own merits, and these clauses would need to be made more explicit and given in greater detail by the Engineer or Architect to suit the proposed scheme.

Testing.—The last eleven clauses cover the methods of testing, both for the materials and for portions of the completed structure. For the purpose of ascertaining

the crushing resistance of concrete it is specified that the test pieces shall be 4 in. or 6 in. in cube. Mr. Etchells stated that the Committee had left this open, but personally he thought 6-in. cubes would give more reliable results, and we agree with him and think it would be quite satisfactory to specify 6-in. cube as the size and eliminate the 4-in. cube. Mr. S. Bylander, as a representative of the Committee, drew special attention to the provisions relating to testing. It was most essential, he said, to define how the tests were to be carried out and also to find the money for doing them. He suggested that the percentage of the contract sum to be allowed must vary according to the size of the job, but probably 1 to 1½ per cent. would be a satisfactory figure. The provision for testing covered by this amount would, of course, not include the manufacturer's tests, which would be paid for by the contractor direct; and he thought that a clear distinction should be drawn in the specification between "Engineer's tests" and "manufacturer's tests."

As to tests of steel, the binding test is undoubtedly the best, and as large-diameter bars of high carbon steel are not considered desirable, 1 in. was suggested as the maximum.

Several other members took part in the discussion, and one or two members sent communications which were dealt with. Mr. R. Colson protested against the use of spacers and chairs in between all longitudinal bars and boarding; and Mr. G. P. Pimm criticised the method described for hand-mixed concrete, and he also dealt with the provision for lifting or rolling piles after maturing. The specification provides that they must be slung or supported on at least two points one-third of the length from each end, and he suggested that it would be better to specify the position of support as one-fifth of the length from each end. Mr. T. C. Dawson wondered why coke breeze should be barred, as he had seen it used in conjunction with rolled steel joists and after thirty years there was not the slightest sign of corrosion.

Mr. H. J. Harding urged the Committee not to put in conditions which would be likely to increase the cost of reinforced concrete, and he also supported Mr. Pimm in his criticism of the specified method for hand-mixing.

Mr. E. P. Wells dealt with the question of hand mixing, and stated that the great secret of proper mixing was to have the aggregates moderately dry, to turn the mixture three times dry, and not trouble so much about the wet. With regard to the points of support in rolling piles, with a 20-ft. pile one need not bother whether the supports were one-quarter or one-fifth of the length from each end. With 70-ft. piles four supports would be better. After dealing with other minor matters he expressed the opinion that, taken as a whole, the specification was exactly what was wanted.

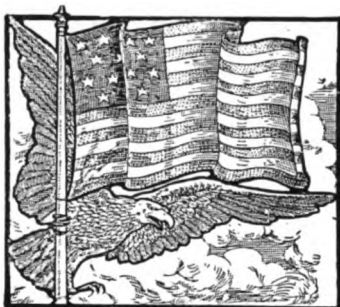
Mr. W. H. Lascelles drew attention to the clause which states that the contractor is to indemnify the employer against any loss by infringement of patent rights, and with which we have dealt in this article.

Mr. Van Osenbruggen said he was not satisfied with the specification as it stood. He did not consider it necessary to include any conditions of contract in a specification, and therefore the whole of Part I. could very well be omitted. In his opinion many of the recommendations went very much too far.

Suggestions were also made by Mr. L. C. Hall, Mr. Warden, Mr. G. S. Roberts, and Mr. A. Scott.

Mr. Etchells, in winding up the discussion, stated that most of the points raised had already been before the Committee, but nevertheless they would be reconsidered in the light of what had now been said.

We hope that the final consideration by the Committee will be given at the earliest possible moment, and the issue of the specification accomplished at a very early date, as the need of such a standard is a very obvious one, and there is no doubt that the excellent work of the Committee will be greatly appreciated by all those connected with reinforced concrete construction.



MAIN CONCRETE DAM AND DIVERSION DAM of MEDINA VALLEY IRRIGATION PROJECT IN TEXAS.

The accompanying illustrations show the construction work on the main concrete dam and diversion dam of Medina Valley irrigation project in Texas. The great concrete dam recently completed near San Antonio, Texas, is 164 ft. high, and contains nearly 300,000 cu. yds.—ED.

THIS irrigation project is based upon the storage of the flood waters of the upper Medina River by means of the main dam, which is located at the head of the cañon through which the stream emerges from a region of rugged limestone hills on to the rolling coastal plains of south-eastern Texas.

It may be stated that the lands to be irrigated lie in a solid body of 60,000 acres in the plain country, at a distance of from 20 to 30 miles from the main dam. The lands are of remarkable fertility, are well drained, and have exceptional transportation facilities, lying along and between two trunk-line railways, the Southern Pacific and the International and Great Northern.

The water is conducted from the main dam to the lands first in the natural river-bed for the four miles of cañon, which will be converted into a narrow lake by means of a secondary or diversion dam. This structure is 50 ft. high, and turns the water into the canal head. From this point the main canal extends 24 miles to the head of the distributing system.

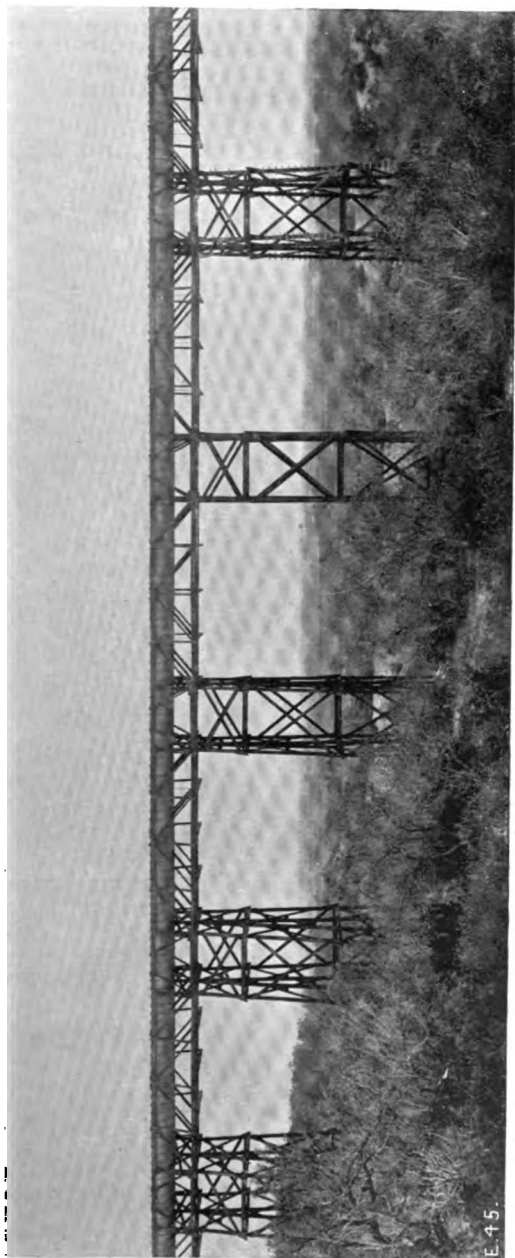
The cubical content of the main dam is nearly 90 per cent. of that of the famous Roosevelt dam near Phoenix, Ariz. The Medina dam is 164 ft. high above the river-bed, and from the bottom of the cut-off trench to the top of the concrete has a total height of 180 ft. The crest length of the dam is 1,580 ft., the base width is 128 ft., and the crest width 25 ft.

The engineer, Mr. Terrell Bartlett, points out that the foundation is in the massive level-bedded limestones, which form the cañon walls and all the hills of the region. The spillway, 1,200 ft. long, is over a saddle into an adjoining ravine. The dam-site is about 14 miles by airline from Dunlay, the nearest point on the Southern Pacific Railway, and its nearest wagon-haul was originally from La Coste, a distance of 23 miles.

As soon as the bonds for the project were sold, excavation for the foundation was immediately commenced, and material for camp buildings, machinery foundations, etc., was hauled in by a new wagon road from Dunlay. At the same time construction was rushed on a standard-gauge railway from Dunlay to the dam.

It is stated that on account of the rough character of the country this line is $19\frac{1}{2}$ miles long, with sharp curves and grades of 3 per cent. against the traffic, and $3\frac{1}{2}$ per cent. against the returning empties. While this railway

was in process of construction, the excavation and lighter preliminary construction, as well as the opening of quarries, were going forward at the dam-site, and simultaneously all the machinery was assembled and heavy timbering was framed at Dunlay.



Flume, Main Canal.
THE MEDINA VALLEY IRRIGATION PROJECT, TEXAS, U.S.A.

In the construction of the project the railroad was completed the latter part of August, and the plant was erected within sixty days thereafter, a really remarkable performance when the unusual magnitude of the plant and its isolation are considered. The materials for making the concrete were all secured at the site with the exception of the cement. The quarries are in a very hard, cherty, semi-crystalline limestone quartz. Sand of a mediocre quality could have been delivered over the railway for about \$1.40 per cu. yd., but it would have been very difficult to handle the large quantity of sand over the single-track, steep-grade road.

It is said that the fine material and dust from the first crushing of the limestone, together with dust from special pulverising crushers, were utilised for sand throughout the dam. The cement used came principally from the mills of three cement companies located in the State of Texas, making as short a haul as possible.

The mixture of concrete used was 1 cement to $3\frac{1}{2}$ sand (limestone dust and fines) to $6\frac{1}{2}$ stone. This required slightly over one

barrel of cement per cubic yard. Daily tests of cubes and of short, plain beams showed a strength of great uniformity. After the dam had reached a height above the zones of heaviest pressures it was, therefore, deemed justifiable to decrease the percentage of cement.

It is held that test specimens with a smaller proportion of cement showed a much greater strength than usual in such mass construction, as well as a high degree of imperviousness. When this decrease was tried in the dam it was found to be detrimental to the plasticity of the concrete and its ease in working, and on this account the attempted economy was abandoned and the original proportion used for the entire dam. Rubble stones up to 10 tons weight were embedded in the concrete. At the bottom from 10 to 15 per cent. of the mass consisted of these plums, but toward the top the percentage gradually decreased. The percentage for the entire dam was 9.9.

The contractor's plant was located on the west bluff, which was dictated by the railway location, which necessarily came in on that side. Adjacent to the end of the railway on one side were the storerooms and the commissary, and opposite were the shops and lumber yards. A spur track on a trestle was carried out over the boiler plant and over the centre of the cement shed, enabling unloading of fuel, oil, and cement by gravity. Power was derived from seven 150 h.p. boilers, with steam lines to the various engine locations. Only the small engine for lighting and the compressors were located at the steam plant.

It must be noted that the two principal quarries were located above the dam on the plant side, the farthest at a distance of 2,000 ft. A third quarry exclusively for plums was located in the east bluff below the dam. Skips of stone and plums from the main quarries were handled by three standard-gauge locomotive cranes on to 4-yd. dump cars. The latter were hauled around a loop track to the crushers by four dinkies.

In this concrete construction work the crushers used were located immediately back of the combined mixer bin and screen houses. Bucket conveyors took the crusher run stone to the screens above the stone and sand bins. The desired percentage of medium sized stone was returned to two disc crushers and two hammer pulverisers for making the sand.

It is pointed out that under the bins was a mixing deck with five hoppers, served by radial cut-off gates from the stone bins and automatic measuring chutes for sand. Cement was delivered to this deck by a belt conveyor from the adjacent shed. On the next deck below was a battery of five 1-yd. batch mixers, emptying into hoppers.

On still a third level below was a narrow-gauge track, on which flat cars carried the concrete buckets about 75 ft. to the cable-ways. This method of delivery was used exclusively for the narrow upper portion of the dam. For the principal mass of the dam delivery was almost entirely by inclined chutes leading from the discharge hoppers down the face of the bluff into a large wooden hopper located over the end of a double-track incline, which extended from bluff to bluff, first along the down-stream edge of the dam, and later at a high level on the face of the dam. For conveying the material the two cableways and control of the Lidgerwood type were used—cables 2½ in., with span of 1,050 and 1,270 ft.; their load capacity was 10 tons.

A car carrying two 2-yd. buckets or 4-yd. per trip was used on each incline track, so that it was possible to send 8 cu. yds. at once to the various derricks by the incline, as well as 2 yds. simultaneously by each of the cables.

The diversion dam shown in the photograph is also of rubble concrete, and is an overflow weir with ogee crest and rollway. It is 50 ft. high, 44 ft. wide at the base, 440 ft. long, and is arched slightly up-stream. The construction plant was similar in general scheme to that of the main dam, except on a much smaller scale, as the diversion dam contains only about one-tenth



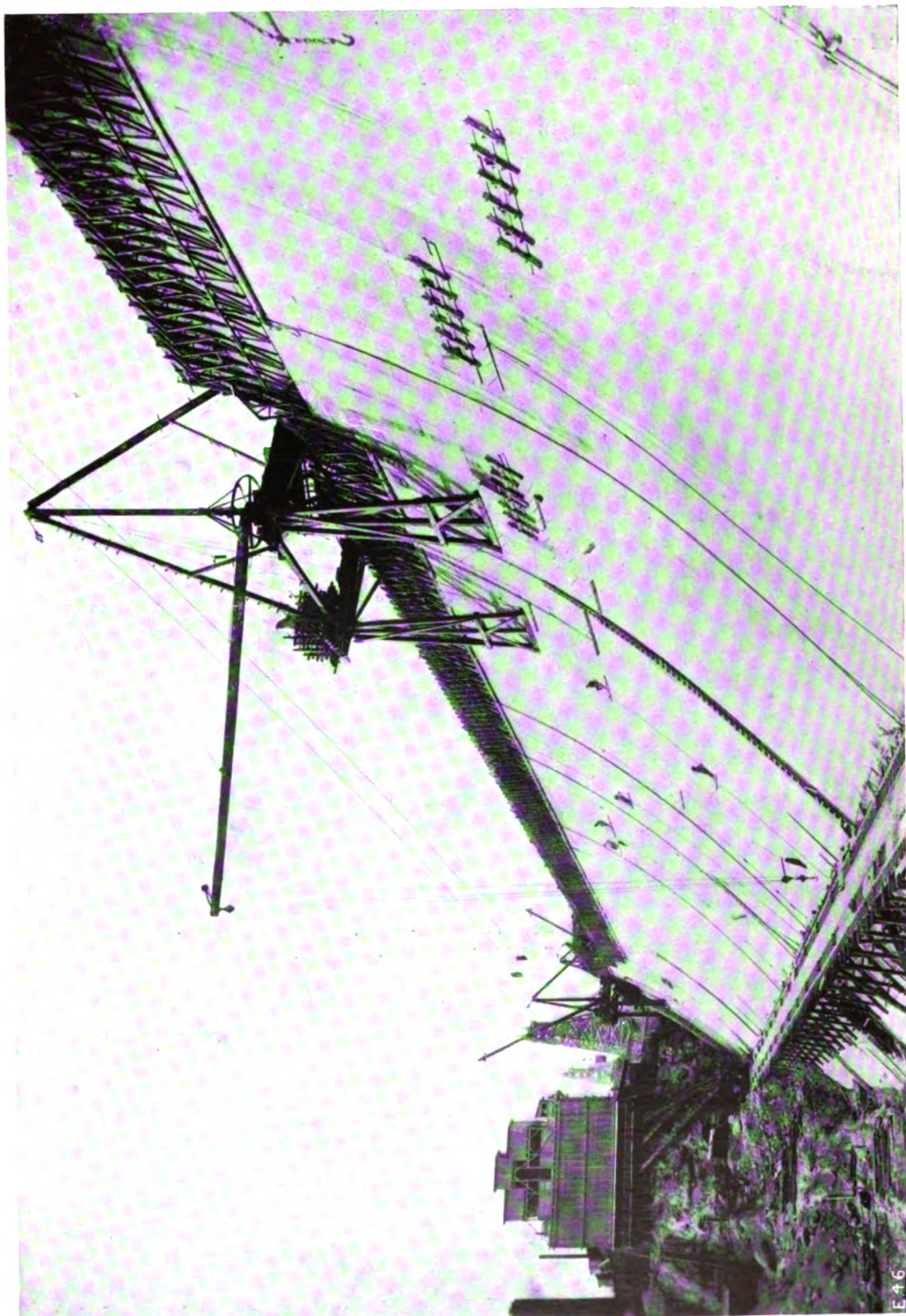
View of Siphon, looking up-stream.
THE MEDINA VALLEY IRRIGATION PROJECT, TEXAS, U.S.A.

of the mass of the former structure. An interesting point of difference was in the utilisation of fixed guy derricks instead of movable stiff-legs.

It may be stated that these derricks were placed on pedestals of concrete within easy reach of each other, and it was possible to so place the guys as to avoid interference with booms or cableways. The lower portion of the pedestals became part of the structure, and the upper part was removed upon completion of the work.

The headworks have a double set of metallic gates. The canal is of ordinary construction, with about 3,000 ft. of concrete-lined channel in rock-cut, the remainder being almost entirely sidehill work in a compact clay.

Considering the reinforced concrete siphons, it is of interest to note the headworks and first mile and a half of the canal are on the west bank of the



View of Main Concrete Dam, facing down-stream.
THE MEDINA VALLEY IRRIGATION PROJECT, TEXAS, U.S.A.

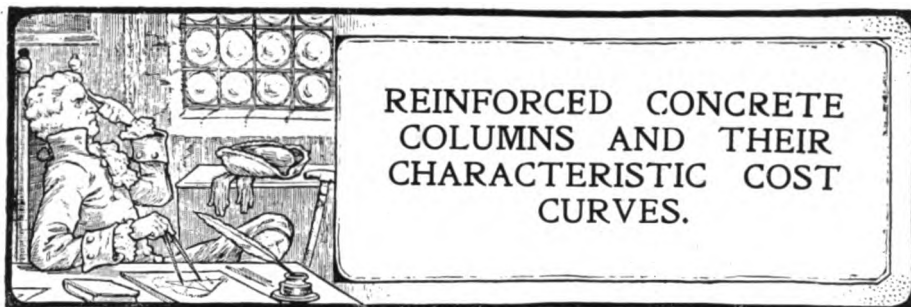
river. The canal is then carried under the river to the east bank in a reinforced concrete siphon, consisting of two bores 8 ft. in diameter. After another mile the canal is returned to the west bank by means of a similar siphon. Two miles of extremely rough country are thus avoided, and the distance shortened about a mile. For conveying the water there are in all eleven flumes along the main canal, aggregating slightly over one mile of flume length. These flumes consist of creosoted timber trestle of long-leaf Texas pine, with truss and tower construction. The foundations are on concrete pedestals. All timber was framed and bored before receiving the 12 lb. creosote treatment. The largest flume is 95 ft. high and 1,500 ft. long.



Headgates and Beginning of Canal at Concrete Diversion Dam.
THE MEDINA VALLEY IRRIGATION PROJECT, TEXAS, U.S.A.

The distribution system is simple and involves no unusual features, except the Chacon dam, an earth structure which will collect return seepage waters from several thousand acres and storm rainfall from about 70 sq. mi., and will also form an equalising reservoir on one of the principal laterals.

It is claimed that an unusual feature of the project is the enormous storage of the main reservoir, which has a maximum capacity of 300,000 acre-feet. This large storage was considered advisable on account of the torrential nature of the stream, and it will make possible the conservation of a large percentage of the run-off from the Medina Basin.



By ROHINTAN N. FRAM MIRZA,
B.Eng. (Lvpl.), A.M.Inst.C.E., M.Inst.M.E., Assoc. Royal T.C.

The following article will probably be of interest to those studying the question of reinforced concrete costs.—ED.

In the present article equations are given to show the variation in the total costs of finished reinforced concrete columns, with respect to the amount of reinforcement employed to increase the resistive power of concrete to compression. The author has already treated the same subject with regard to reinforced concrete prisms subjected to bending, such as beams, slabs, etc. (*vide Concrete and Constructional Engineering*, Vol. IX, No. 2), and that has led him to further investigate the behaviour of costs with varying ratios of steel placed in concrete columns. In all phases of engineering it is always of importance to arrange and proportion the various elements entering into construction so as to bring about the lowest possible sum-total cost. To justify the conclusions it is necessary to give some details of the mathematics used to deduce the same, but several intermediate steps are omitted to restrain the length of the article.

The theory upon which the calculations are based is that set forward by the Joint Committee of the Royal Institution of British Architects and many other well-known authorities. The nomenclature used is the same as that in the Second Report of the Joint Committee of 1911, but with one or two slight alterations and additions as shown below :

$m = \frac{E_s}{E_c}$ = modular ratio, assume m here = 15, and $m' = m - 1 = 14$.

a = area of steel in square inches in compression.

A_c = sectional area of concrete in sq. in. in compression, and A'_c = same in sq. ft.

ρ = ratio of steel to concrete = $\frac{a}{A_c}$.

T_c = total cost. W = imposed load on column in lbs.

c = the allowable working compressive stress on concrete in lbs./sq. in.

t = " " " " tensile stress in steel in lbs./sq. in.

a = one side of a rectangular column in feet,

θa = length of the other side in feet, hence $A'_c = \theta a^2$.

$\theta' = \frac{1 + \theta}{\sqrt{\theta}}$.

S = cost of steel per cwt. on site of work, including placing, etc.

Y = cost of centering per sq. yard on site of work, including placing, etc.

Q = " " concrete per cubic ft. " " " "

$B = [1 + m'\rho]c$, as will be explained later.

The money unit for S , Y and Q may be in shillings, dollars, or any other unit, but it must be the same for S , Y and Q .

Circular centering and round columns being far less used than square or rectangular ones, the latter form will be discussed as the standard example. On p. 17 of *R.I.B.A. Report No. 2* it is given that $W = c[A_c + (m-1)a]$, which can be expressed as $cA_c[1 + m'\rho]$, and which may be again put in an extremely simple form

$$W = 144A_c'B \quad (1)$$

where $B = [1 + m'\rho]c$, or again $\rho = \frac{B-c}{m'c}$.

The value of c adopted throughout is 600 lbs. per sq. in.

$$A_c' = \frac{W}{144B} \text{ and is also } = \theta a^2 \quad \therefore a = \frac{1}{12} \sqrt{\frac{W}{B\theta}}$$

The following abbreviations are used:—

R.C. = reinforced concrete. Max. = maximum. Min. = minimum. T_c = total cost. Sq. = square.

The total cost of a concrete column of unit height is equal to the cost of concrete + cost of steel + cost of centering. The weight of a cub. ft. of steel is 4'35 cwt., the weight of steel used in the column per unit height will be $4'35\rho A_c'$, and obviously the cost will be $4'35A_c'\rho S$ money units. [The money unit need not be particularly in some definite units, but whether it be £'s, shillings, pence or dollars, S , Y and Q must all be considered in the same units.] Hence:—

$$T_c = A_c'Q + 4'35\rho A_c'S + [2a + 2a\theta] \frac{Y}{9} \quad (2)$$

Substituting values of A_c' , ρ and a in terms of B

$$T_c = \frac{WQ}{144B} + 4'35 \frac{WS}{144B} \cdot \frac{B-c}{m'c} + \frac{1}{54} Y \sqrt{\frac{W}{B}} \left[\frac{1+\theta}{\sqrt{\theta}} \right] \quad (3)$$

and differentiating T_c with respect to B

$$D(T_c) = \frac{WQ}{144} \left(-\frac{1}{B^2} \right) + \frac{4'35 WS}{144m'B^2} + \frac{\theta'}{54} \frac{\sqrt{W}}{2} \times \left(-\frac{1}{B^{\frac{3}{2}}} \right) Y \quad (4)$$

again to determine that value of B , which will make T_c a max. or a min., it is necessary to equate equation (3) to zero and then solve for B , hence it follows that

$$B = \frac{\theta}{1+\theta} \left[\frac{31S-Q}{Y} \right]^2 W \frac{\theta}{(1+\theta)^2} \quad (5)$$

For θ = unity, i.e., when the column is square

$$B = 14 \left(\frac{31S-Q}{Y} \right)^2 W$$

Noting that value of ρ which makes T_c a maximum by suffix c and remembering that $B = [1 + m'\rho]c$

$$\rho_c = \left[\frac{\frac{\theta}{1+\theta} \left[\frac{31S-Q}{Y} \right]^2 W \theta}{14c} - c \right] \quad (6)$$

Equation (5) gives all the information necessary to study the effect of varying ρ on T_c , but the investigations would not be complete without noting the effect of varying W on T_c ; this may best be shown by actually working out two examples with some actual data.

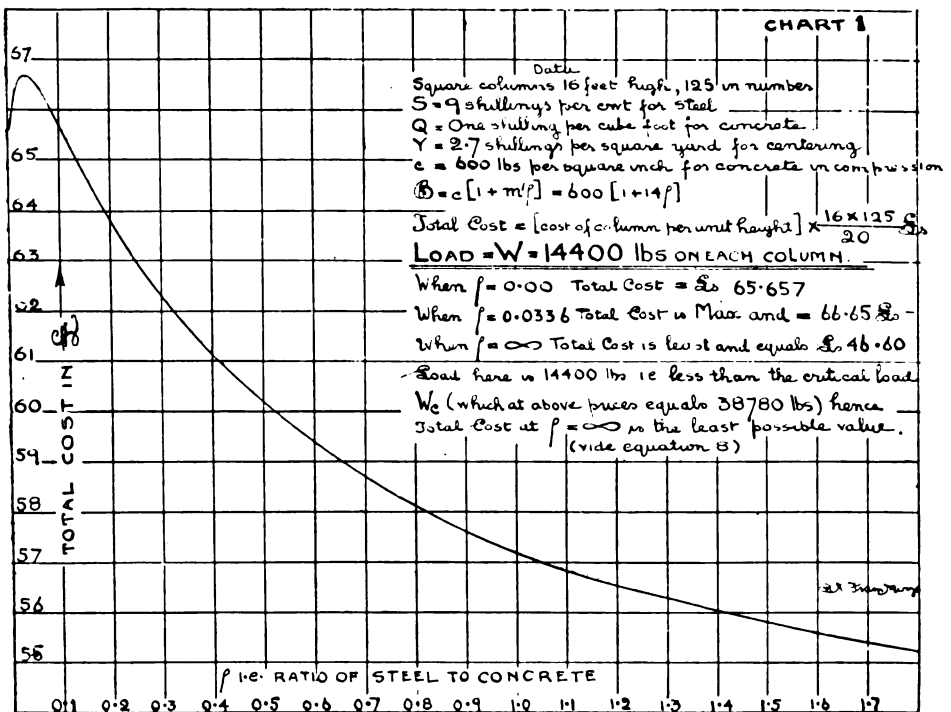
EXAMPLE I. 125 square (i.e., $\theta = \text{unity}$) columns. $W = 14,400$ lbs. on each column. $c = 600$ lbs./sq. in. $S = 9$ s./cwt. $Q = 1$ s./cub. ft., and $Y = 2.7$ s./sq. yd., money units being in shillings. Substituting these values in equation (3), T_c equation reduces to

$$T_c = £100 \left(466 - \frac{1796}{B} + \frac{12}{\sqrt{B}} \right) \quad (7)$$

The total costs corresponding with various values of ρ (where $\rho = \frac{B-c}{m'c}$) have been plotted in Chart No. 1.

From equation (5) $\rho_0 = .033$ and the chart also shows the same.

It is important to note that the value of T_c at $\rho = 0$ is $>$ than that at $\rho = \infty$.



EXAMPLE II. In this case let $W = 90,000$ lbs. and the other conditions remain the same as in Example I.

ρ_c in this case works out as 0.588, as also seen in Chart 2, in which the total costs corresponding to various values of ρ are plotted, with regard to this example.

Here it should be noted that T_c at $\rho = 0$ is $<$ than at $\rho = \infty$.

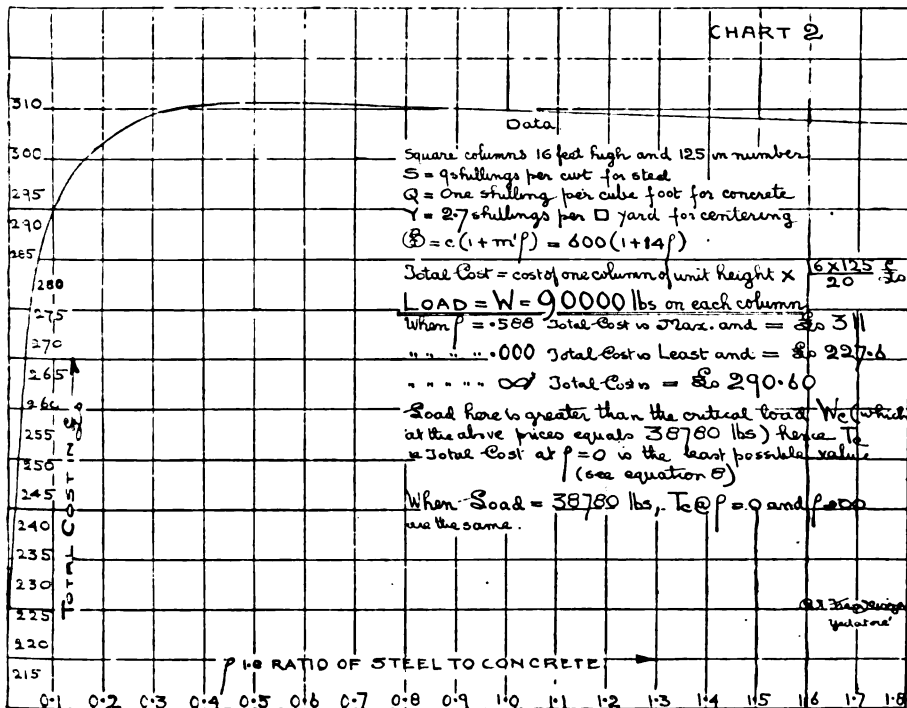
This leads to a very important result, for, given the values of S , Q and Y there must be one value of W which gives T_c at $\rho = 0$ and $\rho = \infty$ exactly the same and this critical value of W can be obtained by equating the value of

equation (3) at $\rho=0$ to the value of the same equation at $\rho = \infty$. Denoting this critical value of W by symbol W_c

$$W_c = \frac{64 \times Y^2 \times c(1+\theta)^2}{9(31S-Q)^2\theta} \quad (8)$$

Solving this for values of S , Q , Y and θ , as given in Examples I and II, $W_c = 38,780$ lbs., or about $17\frac{1}{2}$ tons, hence
for values of imposed loading (on one column) lower than that given by W_c , the least possible total cost of that column will be when $\rho = \infty$ (this is possible in the case of an all-steel column), and similarly for values of imposed loading greater than W_c , the least possible total cost will be obtained when $\rho = 0$.

Curves in Charts 1 and 2 give graphic interpretations of the above statements.

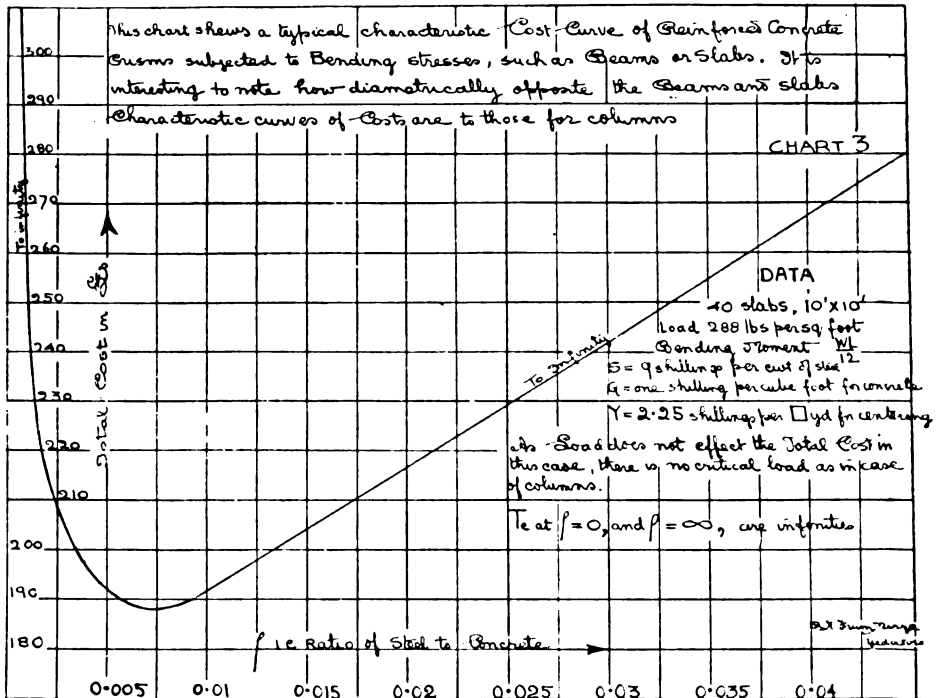


Conclusions.—(1.) The total cost of columns having the same resisting strength varies with the ratio of reinforcement, and rises to a maximum when ratio of reinforcement is equal to the value given by the expression $\left\{ \frac{9}{16} \left[\frac{31S-Q}{Y} \right]^2 - \frac{W\theta}{(1+\theta)^2} - c \right\} \div 14c$, where S =price of steel per cwt., Q =price of concrete per cub. ft., Y =cost of centering per yd. All the prices must represent the final price after fixing in position, etc., at the site of work, and must be in same money units, that is, if S is in shillings, Q and Y must also be in shillings. If the column is rectangular θ is the ratio of one side to the other and this ratio is unity for a square column. c is the compressive strength of concrete in lbs. per sq. in., and is usually taken as =600 lbs./sq. in.

(2.) For values of W , i.e. imposed loading (on one column) $<$ than that given

by expression $\frac{64}{9} \frac{Y^2}{(.31S-Q)^2} \frac{(1+\theta)^2}{\theta}$, the T_c of that column with nil reinforcement is $>$ than the T_c , when $\rho = \infty$ (rising of course to a max. between the extreme values of 0 and ∞), similarly when W is $> \frac{64}{9} \frac{Y^2}{(.31S-Q)^2} \frac{(1+\theta)^2}{\theta}$ the total cost is lowest at $\rho = 0$. For square columns and $S = 9s./cwt.$, $Q = 1s./cwt.$ and $Y = 2.7$ shillings per sq. yd., this critical value of W is 38,780 lbs.

(3.) For conditions considered the calculations give true results, but the two following facts must not be lost sight of: (a) the superior fireproofing qualities of concrete as compared to all-steel columns (which would be the case when $\rho = \infty$)



and again (b) that columns with no reinforcement (which would be the case when $\rho = 0$) occupy greater space. ρ stands here for the ratio of steel sectional area to that of concrete.

(4.) For small values of W , the difference between min. and max. T_c is not great, but difference increases rapidly as W reaches higher values.

(5.) Hooping a column artificially raises the value of c and for calculations c will be different to 600 lbs./sq. in., thus making a corresponding change in the deduction of the values of ρ_0 and W_c , when calculating the same for hooped columns.

(6.) The results are surprisingly different from those obtained by the author for Cost Curves of R.C. beams, etc. (vide *Concrete and Constructional Engineering*, Vol. IX., No. 2), in which case the T_c curve with respect to ρ assumes a min. value for a certain critical ratio of reinforcement. Such a characteristic curve is shown is Chart No. 3.



Fig. 1. Some Finished Cottages (back).

CONCRETE COTTAGES AT CRAYFORD.

*The following particulars
will doubtless be of interest
to those concerned in the
question of rural housing.*
—ED.

SOME very good examples of cottages constructed with concrete blocks are provided in the scheme which is now proceeding for the Crayford Cottage Society in Kent, under the direction of the Rural Housing Organisation Society, and some particulars of the design and construction should prove of interest.

The estate that is being laid out is a fairly large one, and the scheme includes 457 cottages and five sites for public buildings, while portions of the land will be given up to a recreation ground, village green, bowling green and various allotments, the last-mentioned being in addition to the large gardens that are to be provided to each cottage as shown in Fig. 3.

The site adjoins the River Cray, and is quite close to the village of Crayford and but a few minutes' walk from the extensive works of Messrs. Vickers; thus it will be convenient to many of the workers in the latter. The concrete cottages are being executed by the Cottage Construction Co., of 16-17, Devonshire Square, E.C., and they form the best types of this class of work in the vicinity of London, and an interesting comparison is afforded between concrete blocks and brickwork, as regards economy and speed of erection in cottage work, which tends to show the former to advantage.

The same type of plan has been adopted for the whole of the cottages now under construction and completed, but the elevations are varied to avoid monotony. Each dwelling has a frontage of 24 ft. 4½ in., and a depth of 17 ft. for the main building, with a projection of 6 ft. for a length

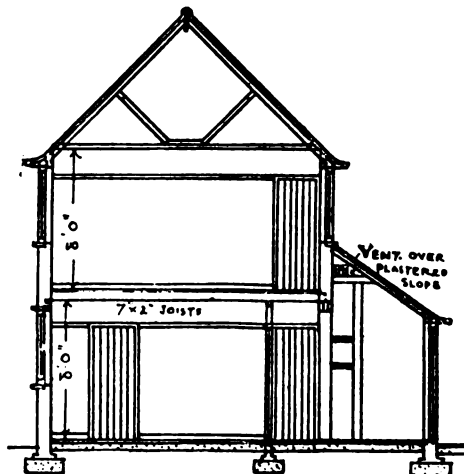


Fig. 2. Cross-Section.

CONCRETE COTTAGES AT CRAYFORD, KENT.



Fig. 3. Plan showing Lay-out of Cottages.
CRAYFORD VILLAGE EXTENSION, KENT.

of 12 ft. at the rear of the ground floor. The accommodation on the ground floor consists of a living room 15 ft. 6 in. by 11 ft., a parlour 12 ft. 4½ in. by 9 ft., a scullery, containing bath, a larder, w.c., and fuel store, and on the first floor three bedrooms are provided, the largest being 15 ft. 6 in. by 9 ft., and the smallest 8 ft. 3 in. by 7 ft. 9 in. Each floor has a height of 8 ft. in the clear, and the disposition of the rooms is shown in the plans illustrated in Figs. 4 and 5.

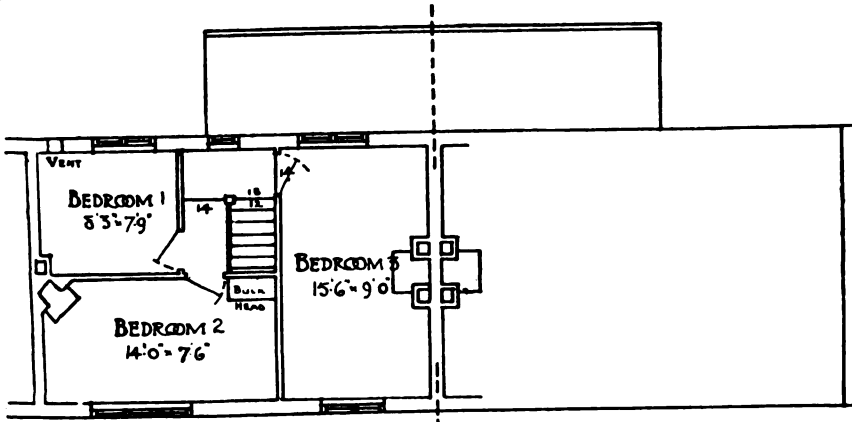


Fig. 4. First Floor Plan.

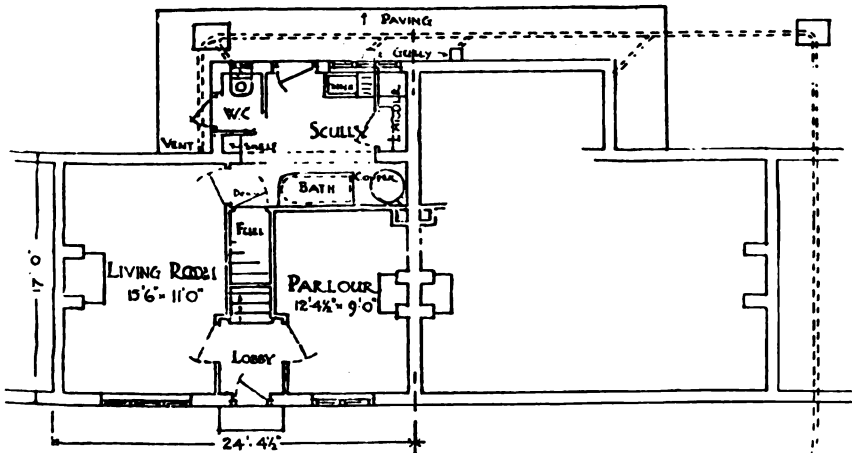
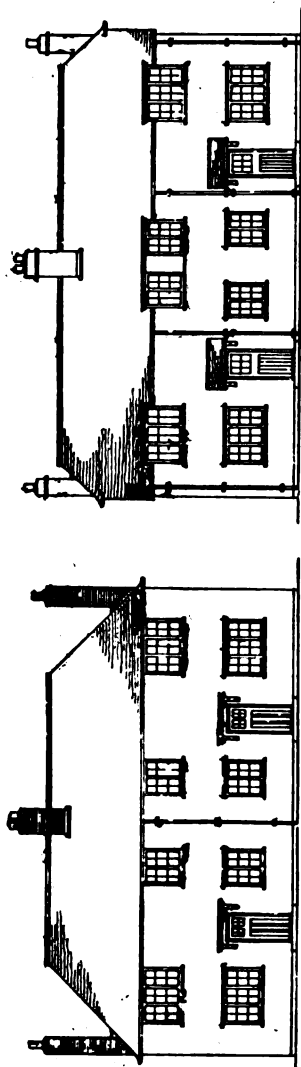


Fig. 5. Ground Floor Plan.
CONCRETE COTTAGES AT CRAYFORD, KENT.

The contract cost per cottage erected with concrete blocks, without extras, works out from £175 to £179, and it is interesting to note that the same contractors' price for the brick cottages was £15 more than this per cottage. The difference in cost amounts to a considerable sum when applied to the whole scheme, and there is no doubt that with the methods employed in this instance the concrete block construction is superior to the brickwork. The extras refer to gables and hipped roofs necessary to secure external variation. The cost of these is proportionally moderate.



Types of Front Elevation.

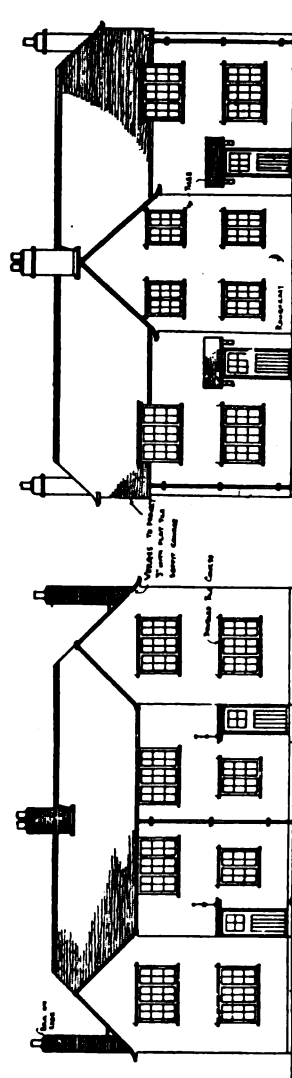
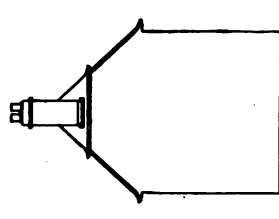


Fig. 6. Types of Front Elevation.
CONCRETE COTTAGES AT CRAYFORD KENT.



End View.

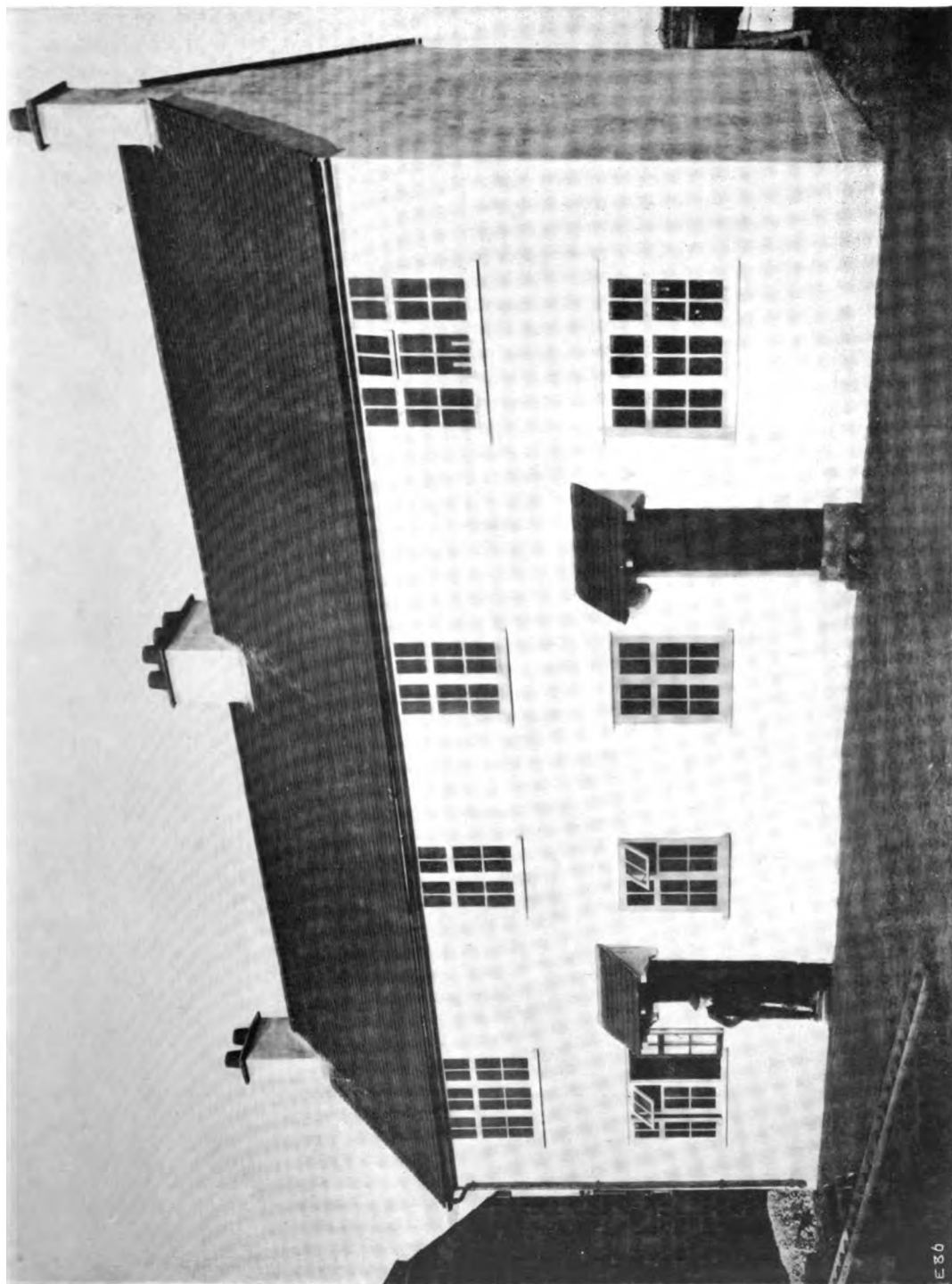


Fig. 7 Pair of Cottages after Completion.
CONCRETE BLOCK COTTAGES, CRAYFORD, KENT.

At the present time definite arrangements have been made for 150 concrete cottages, and of this number some are completed and occupied, and no less than fifty will be completed and ready for occupation in three months from the commencement of building. This speaks well for the speed of erection, and will not be equalled in the case of the brick cottages which are being erected on the same estate.

The external walls are constructed with a thickness of 9 in., the concrete blocks being 9 in. by 9 in. by 16 in., and they are built directly on to the concrete



Fig. 8. Blocks laid out for Seasoning.



Fig. 9. Cottages during Construction.
CONCRETE COTTAGES AT CRAYFORD, KENT.

foundation, which is about 2 ft. wide and 9 in. thick. The internal divisions are all built with $2\frac{3}{4}$ in. breeze concrete partition slabs made on the same machines as the concrete blocks. The whole of the chimney breasts, flues and stacks are built with the blocks, blocks being moulded for the flues, and

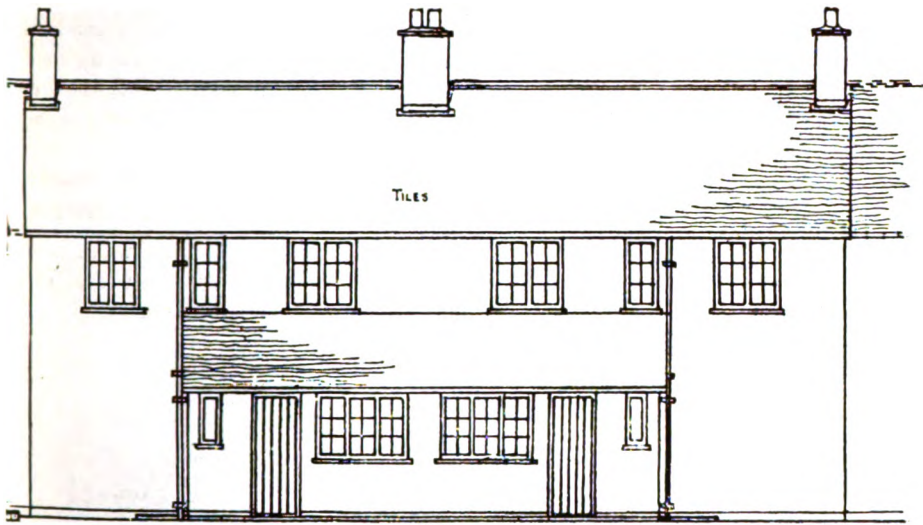


Fig. 10. Back Elevation.
CONCRETE COTTAGES AT CRAYFORD, KENT.

different sizes being employed to give the necessary bond. The gables are also finished at the top course with blocks having one end splayed to suit the pitch of the roof, and by this method the "racking back" is performed without the introduction of small filling pieces, which are generally employed in inferior work. The window and other openings are generally spanned by a wood lintel, but relieving arches composed of the blocks are built over same to



Fig. 11. A Cottage before Rough-casting.
CONCRETE COTTAGES AT CRAYFORD, KENT.

take the superimposed load and transmit it direct to the wall on either side of the openings. Splayed blocks were used in many cases to avoid sharp angles at the sides of openings in the interior of the building, and no trouble has been spared in those minor details which affect the character of the finished structure. The bedroom floor is constructed with ordinary wood joists boarded on top and plastered below, and the ground floor is finished with boarding fixed to splayed fillets bedded in the surface concrete.

The whole of the exterior walls and chimney stacks are rendered with cement and sand containing a waterproofing material, and finished with rough-cast, thus eliminating any possibility of dampness penetrating the walls, these being only 9 in. thick, and not built of blocks moulded with concrete gauged to give the maximum resistance to percolation of water. It is well known that



Fig. 12. A Group of Four Cottages.
CONCRETE COTTAGES AT CRAYFORD, KENT.

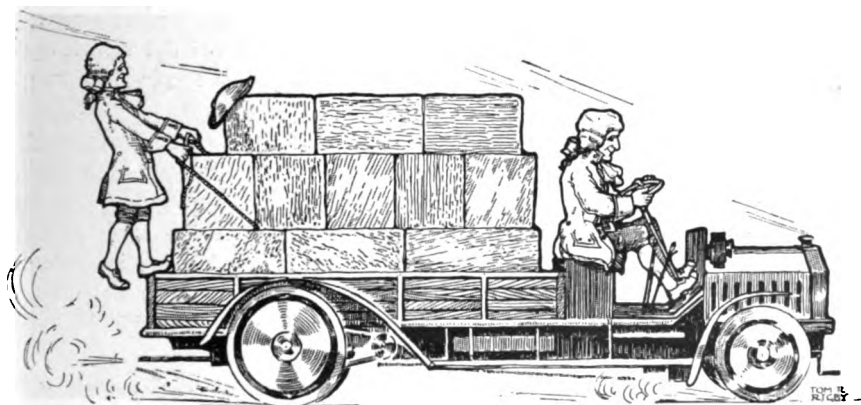
brickwork 9 in. thick is not sufficiently weather-resisting to keep out the damp unless built with a cavity, and the formation of the latter increases the cost, and generally causes the whole of the weight from the floors, etc., to be thrown on the inner leaf of brickwork, which is only $4\frac{1}{2}$ in. thick. The whole of the interior walls are plastered with the exception of the scullery, in which the walls are finished with distemper applied directly to the concrete blocks, and the appearance is very satisfactory.

Various photographs of the work at different stages and after completion are given in the photographic views, and it will be seen that the appearance is quite satisfactory from the artistic point of view, although the elevations are extremely simple. It will, of course, be obvious that, in cases where the walls are finished with rough-cast, as here described, the appearance is the same as that obtained with brick buildings which are treated externally with the same

material, and the concrete block construction is cheaper and undoubtedly stronger.

The blocks employed in the work are made with a mixture consisting of five parts of ballast and one part of Portland cement. The ballast is obtainable in the vicinity of the site, and merely requires screening to remove any large stones which will not pass a $\frac{3}{4}$ -in. mesh, as it is very clean and contains a satisfactory amount of good sand. The blocks are moulded semi wet and allowed to season for a minimum period of fourteen days before being built in the work, and they have a much longer period when in position to harden before being covered up. No wetting of the blocks has been necessary during seasoning up to the present, as the weather has been excessively wet, and in fact the work has been proceeding during the worst possible weather conditions, and this is an important factor when the great speed is considered.

The blocks have also required to be protected during the frosts, and in spite of adverse conditions they are very sound and strong. It is roughly estimated that 80,000 blocks were required for the first fifty cottages, and the whole of these have been made on the site with four machines supplied by Winget, Limited, 25, Victoria Street, Westminster, S.W., and Newcastle-on-Tyne, the type of machine being so well known as to need no special description.





CONCRETE ROAD CONSTRUCTION.

The following notes on Concrete Highway Construction have been prepared for us by Mr. A. N. Johnson, the State Highway Engineer of Illinois, U.S.A.—ED.

GENERALLY.

THE use of water-bound macadam and gravel roads has been very common throughout the United States, and for a number of years served all needs of traffic, but with the increase in the number of motor vehicles using the roads the cost of maintenance increased very rapidly. This led engineers to try other forms of construction which would be more durable under motor traffic.

In the States where the soil is a heavy black loam, common to the so-called Corn Belt, the waterbound roads also suffered considerable damage from the sticky mud which was carried on to the surface of the road by traffic. At certain times of the year the consistency of this mud is such as to render it very sticky, with the result that it would adhere to the wheels of the vehicles and also to the stones in the road, pulling them out and making the surface rough and uneven.

In order to prevent the damage done by this mud and by motor traffic to waterbound roads, many engineers have resorted to the use of a bituminous binder for the surface of the crushed stone roads.

Under some conditions this construction seems to be very satisfactory, but in the Corn Belt it is found that the bituminous binder is more or less injured by the sticky mud which is carried on to the road surface, so that while this form of construction is satisfactory so far as motor traffic is concerned, it does not entirely prevent damage by mud.

These conditions have led engineers to try other forms of construction, and among those that have been developed is the concrete road.

THE CONCRETE ROAD.

In the work of the Illinois Highway Commission, the concrete roads have been built of widths varying from 12 ft. to 18 ft. The 12 ft. and 16 ft. roads have been made 6 in. thick, while the wider roads have been made 8 in. thick at the middle and 6 in. thick at the edge. The surface is constructed with a cross slope of $\frac{3}{8}$ in. to the foot. In order to prevent dislodgment of the slab of concrete by the freezing of water which might run in under it, underdrainage is provided.

Drainage.—The underdrainage consists of longitudinal trenches about 8 in. wide and 6 in. deep, constructed along each edge of the pavement, the trench being filled with broken stone. In addition to the longitudinal broken stone

drains, cross drains are constructed every 50 ft., running from the longitudinal drains to the gutters. These cross drains are also filled with broken stone to within four or five inches of the surface of the side road, and then covered with earth.

Aggregates.—The aggregate for the concrete may be either broken stone and sand or properly graded natural gravel, and it is believed that the gravel is much superior to the broken limestone, since the particles of gravel are hard and will wear much longer than the broken limestone. If trap rock or granite, or equally tough rocks are available for the aggregate, they are satisfactory.

It is important that the particles of coarse aggregate near the road surface shall be completely surrounded with cement mortar, so that there will be no possibility of dislodgment of the larger stones under traffic. If individual stones are dislodged from the surface, a break is started which will increase gradually



CONCRETE ROAD CONSTRUCTION IN U.S.A.

under traffic, until it becomes of a size requiring repairs, if the road is to be preserved. For that reason the mixture of the concrete should be such as to provide an excess of mortar, and consequently the mixture of one part cement, two parts of sand, and three and one-half parts of gravel pebbles or broken stone is used. The surface of the road should be somewhat granular, so as to afford a good footing for horses, and should, therefore, be finished with a wood float which should be used merely to flush sufficient mortar to the surface to surround thoroughly the larger pebbles in the aggregate.

Precautions against variation in temperature.—In order to take care of the change in length of the concrete pavement under variations in temperature, expansion joints are provided every 50 ft., and it has been noted that the weakest place in the concrete surface is at the expansion joints. The steel tired wheels rolling across the joint have a tendency to break down the concrete, and

unless protected from this sort of damage, the pavement deteriorates rather rapidly at the joints. Several methods are being employed to prevent this damage. One plan is to use a pair of steel plates, one being attached to each of the slabs which come together at the joint. The plates are separated about one-half inch when the road is constructed, and the opening filled with some compressible material, such as asphalt.

Another method is to use a row of creosoted wood paving blocks at the joint.

General Construction.—In constructing the roads the following general method is being used by the Illinois Highway Commission. The road-bed throughout a considerable portion of the length of the road is shaped and rolled, and the aggregate dumped on this prepared road bed in approximately the quantities necessary for the concrete. The mixer used is one which takes



CONCRETE ROAD CONSTRUCTION IN U.S.A.

the material in at the front end and delivers it at the rear. The mixing drum is loaded by means of a chute which is let down on the ground for filling, and is then raised until the aggregate is dumped into the drum. The concrete is distributed on the roadway by means of a bucket which travels along a boom at the rear of the machine, and can be automatically dumped at any place desired. The boom is adjustable as to height and can be swung in a circle across the roadway. The machine is mounted on wheels and moved along the road under its own power. The various processes in the construction are best shown by the accompanying photographs.

ROAD BRIDGES.

In considering the development of any considerable mileage of American highways, the bridging of waterways must be given careful consideration.

In the State of Illinois practically all of the bridges and culverts built prior

to 1907 are either temporary wood structures or light steel structures provided with wood floors. It has been found that on account of the excessive and constantly increasing cost of maintenance, it is highly important that permanent bridges be built, wherever such structures are economically possible.

Reinforced concrete slabs for openings, varying in width from a few feet up to 25 or 30 ft., are particularly adaptable to country highway conditions. The principles of construction are simple enough, so that it is not difficult to find men competent to build them, whereas, in arch structures and the more complicated forms of beam and girder construction, the principles of design are more or less involved and require care in construction not easily obtained.

Comparing simple slab designs with beam and girder designs, the economical limit of span length as regards materials is considerably under 30 ft. for simple slabs, but owing to the increased difficulty of securing workmen competent to build the more complicated designs, there is but little saving in using the beam and girder design for spans shorter than thirty ft., unless concrete materials at the bridge site are exceptionally costly. On the other hand, where concrete materials are exceptionally cheap, simple slabs can be used to advantage for longer spans. Simple slabs have been economically used for highway bridges having clear spans of 42 ft.

For longer spans reinforced concrete "deck" or "through" girder bridges may economically be built with clear span lengths of 60 or 70 ft. In designing such bridges it is equally important from considerations of economy and good workmanship to make the design as simple as possible.

As the span length or the cost of materials on the ground increases, it becomes more and more important to avoid the use of surplus material. It is rarely advisable or economical, however, for the ordinary country highway bridge to make the design complicated, such, for instance, as results from the use of floor beam and slab construction, for the floor system.

It is true that by lightening the floor system, as much as possible, by using floor beams, longer girder spans are theoretically possible, but the conditions at the bridge site are ordinarily such as to make it more economical to use two or more shorter spans rather than one long one. For instance, a 70 ft. reinforced concrete superstructure with a 16 ft. roadway would cost, under average conditions, about \$2,600.00. Under the same conditions, two 35 ft. reinforced concrete girder superstructures would cost about \$1,500.00. The two span bridge would, therefore, be cheaper, provided the pier could be built for less than \$1,100.00, which would usually be the case.

Arch Construction.—Under proper conditions the arch may well be used in place of slabs or girders. Where rock or other foundation of unchangeable character may be obtained within a reasonable distance below stream bed, and where there is plenty of head room and the span is long, the arch may prove to be the economical design to use.

Slabs or girders are better adapted, however, to conditions of ordinary head room and for foundations requiring piles, or which may permit settlement of the abutments.

The advantages of the slab or girder design over the arch are due chiefly

to the following facts: The pressure on the foundation is vertical and no precaution need to be taken to avoid sliding. The weight of the structure complete is much less, requiring less costly foundations. With the superstructure separated from the substructure, no damage whatever occurs from a considerable settlement of either or both of the abutments, whereas the settlement of one of the abutments of an arch structure would produce unsightly cracks and dangerous stresses.

Expansion rockers can easily be provided so that there are no temperature stresses in the superstructure. The effect of changes of temperature is too often neglected in arch design. Lastly, the area of waterway may be greater for the same length of span.



CONCRETE ROAD CONSTRUCTION.



*It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.
The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.*

THE CONCRETE INSTITUTE.

SOME MODERN METHODS OF ARCH CALCULATION.

By EWART S. ANDREWS, B.Sc., M.C.I.

The following is a short abstract of a Paper read at the 55th Ordinary General Meeting of the Institute.

THE arch is a form of structure which possesses great advantages from the standpoints of beauty and economy, and from the earliest times the arch has been used in all kinds of constructional work. In the present paper arches are not considered at all from the point of view of architectural styles or orders, but consideration is restricted to the calculation of the stresses in them.

The arch presents points of considerable difficulty from this point of view, and the resulting formulæ are elaborate. Some may contend that the formulæ are too elaborate, and that "simple practical rules" are just as good; one answer to such contention is that such simple rules would be welcomed if they really were as good. The difficulty is that, unless simple rules are applicable over a wide range, and have been fully tested by scientific experiment, there is considerable danger in their use.

There is, unfortunately, among some practical engineers a strong antipathy to complicated formulæ—an antipathy which is usually stronger in proportion as the formulæ are not understood—but closer acquaintance with such formulæ and some useful spadework in the form of the compilation of tables and diagrams, usually helps to dispel much of the dread. The primary reason of the difficulty in the determination of the stresses in arches arises from the fact that, in most cases, the arch is what is called "a statically indeterminate structure," so that the forces acting upon the arch cannot be found by the ordinary laws of statics. Exactly similar difficulties arise in the case of stiffened suspension bridges, continuous beams, and slabs.

The stresses in an arch can be found as soon as we can find the magnitude and position of the reactions; these reactions (R) may be considered as compounded of vertical components (V) and horizontal thrusts (H); if, as is usual, all the loads on the arch are vertical, the horizontal thrust (H) must be the same at each end.

We can then draw the line of pressure, linear arch, or equilibrium polygon (three alternative names for the same thing) for the given load system upon the arch.

Then, by Eddy's theorem that "the bending moment at any point of an arch is equal to the product of the horizontal thrust into the vertical intercept between the centre line of the arch and the line of pressure," we have—

$$\text{Bending moment at any point} = P = H \delta$$

If the line of pressure comes below the centre line of the arch, the upper surface or extrados of the arch will tend to become in tension; and if the line of pressure is above the centre line of the arch, the lower surface of intrados of the arch tends to become in tension.

Stresses in the Arch.—To obtain the stresses in the arch we first find the thrust or normal pressure (Q) and the shearing force (S) at the point by resolving the thrust (O_3) at the point under consideration along and perpendicular to the centre line of the arch at the given point.

Then, if A is the area of the section, and M_c , M_t the compression and tension moduli, we have—

$$\text{Maximum tension stress} = t = \frac{H\delta}{M_t} - \frac{Q}{A} \quad (1)$$

$$\text{Maximum compression stress} = c = \frac{H\delta}{M_c} - \frac{Q}{A} \quad (2)$$

$$\text{Mean shear stress} = s = \frac{S}{A} \quad (3)$$

There is therefore no difficulty whatever in the calculation of stresses in arches when once we know the horizontal thrust.

Cases in which the horizontal thrust can be found without the elastic theory of arches: these are as follows:—

1. Parabolic arch uniformly loaded over whole span—

$$H = \frac{wl^2}{8v}$$

w = load per unit length,

l = span of arch,

v = rise of arch.

2. Parabolic arch with uniform load extending from an abutment to the centre—

$$H = \frac{wl^2}{16v}$$

3. Arch of any shape provided with hinges. In this case the line of pressure can be drawn by the well-known graphical construction for making a link-polygon pass through any three given points; then the polar distance of the vector diagram gives the horizontal thrust desired.

This theory, to which the author's principal attention was given, is based upon a consideration of the deformations which an arch-rib obeying the ordinary laws of bending will receive. From these deformations we are able to calculate the horizontal thrusts.

Let δs represent a very short length between two points of an arch-rib, one end of which is considered as fixed relatively to the other end, and suppose that the bending moment along this very short length is B .

If E is the elastic (Young's) modulus and I the moment of inertia of the rib, it can be shown by the theory of beams that—

Δy = vertical displacement of the end due to bending

$$= \sum_A^D \frac{Bx\delta s}{EI} \quad (1)$$

Δx = horizontal displacement of the end due to bending

$$= \sum_A^D \frac{By\delta s}{EI} \quad (2)$$

$\Delta \theta$ = angular change of tangent to rib at end

$$= \sum_A^D \frac{B\delta s}{EI} \quad (3)$$

These are the general formulæ upon which all the special formulæ are based, and it should be noted in passing that they are the deformations due to bending only and do not include those due to direct thrust; we will refer to the effect of the latter at a later stage.

These general formulæ were then applied to the most common special cases of rigid arches, viz. :—

- (1) Arches with two hinges or pin joints, i.e., two-pinned arches.
- (2) Arches without hinges, i.e., fixed arches.

In a Paper of this kind it was manifestly impossible to deal fully with a subject upon which several long text-books have been written, and it might very well be that many important points have been omitted. The author hoped, however, that a study of the Paper would enable those who had not had the opportunity of a complete study of the arch theory in its more general aspects to follow the fundamental relations upon which scientific calculations were based, and that their interest would be sufficiently aroused to encourage them to study the subject more fully.

THE SOCIETY OF ARCHITECTS.

THE CONSTRUCTION AND PROTECTION OF BUILDINGS IN RELATION TO FIRE.

By A. ALBAN H. SCOTT, Vice-President.

At a meeting of the Society of Architects held last month an interesting Paper was read by Mr. Alban Scott on the Construction and Protection of Buildings against Fire. We give below an abstract as far as the Paper dealt with constructional matters. The latter portion, not reproduced here, dealt mainly with equipment and fire extinction.

Introductory.—With all the means at our disposal to-day for preventing and fighting fires we still have appalling loss of life and damage to property through fire. Every year improvements are made in all branches to prevent such loss, and again every year seems to bring forward also more hazardous trades, more concentration and other causes of large loss.

In giving a paper of this description it would be impossible to do so without referring to that excellent institution, the British Fire Prevention Committee, with which two of our members, namely, Mr. Edwin O. Sachs and Mr. Ellis Marsland, have been so long associated, and have taken such an infinite amount of care and devoted so much time to its work. Although I have been a member of the British Fire Prevention Committee for some period and duly receive from time to time their valuable publications, it is only when one commences to study them in detail that one fully realises the enormous ground such a Committee cover; from the testing of flannelette up to the testing of heavy reinforced concrete floors, and special commissions and reports on what our friends in France and Russia have done in fire precautions.

London Building Acts.—In reading through the London Building Acts with fire precautions specially in mind, it is pleasing to note what great care has been taken generally regarding the construction of buildings from the fire point of view. In spite of all these precautions, I think additional provision for escape of occupants in case of fire should be provided and the powers given to the Council still further increased in this respect.

It would be advisable for the London Building Acts, and in fact all our by-laws throughout the country, to make it compulsory that *all* external walls and internal partitions and floors are constructed of fire-resisting material. This sounds perhaps rather a startling proposition, but if we just consider for a moment the following figures, it does not perhaps mean quite such a drastic expense as one would think from an outline statement.

The whole of the following costs have been based on the current cost of materials.

	Per ft. sup. Pence.
<i>Floors.</i> Wooden floors suitable for domestic buildings taken in spans of 12 ft.	11'3
Fire-resisting floor, ditto	17'3
Wooden floors for workshop at 200 lbs. super load per square foot. Spans 10 ft.	28'9
Ditto, ditto. Fire-resisting floors	34'4
(Basis of last two items taken for floor 30 ft. by 100 ft.)	

										Per yd. sup. Pence.
Cost of wood framed partition, plastered on both sides, per yard										
super	...	...	...	...	...	...	...	...	...	72'8
Cost of fire-resisting partition, plastered on both sides										72'0

It would, I think, be a great mercy to the building owner, the community generally, and to architects if the London Building Acts were in force through the country (with certain modifications), and administered by district surveyors of the same standard and training as in London.

I do not propose to touch upon all the points in these Acts which relate directly or indirectly to fire prevention in its various aspects. The widths of streets and frontage lines indirectly bear upon the point as well as courts within a building and space at rear of domestic buildings. As to the courts, I would like to see introduced that where the opposite walls of courts are less than a certain distance apart all openings in the walls should be protected by fire-resisting wired glass and hard metal frames and sashes.

Clause 62 should be so amended that all floors, irrespective of height, should be constructed of fire-resisting material.

Clause 64. The word "stone" should be deleted and corbels for all chimneys should be supported on fire-resisting material other than stone or hard timber.

As indicated elsewhere, I suggest Clause 68 should read that all floors and staircases, irrespective of the size or use of the building, should be constructed of fire-resisting material, and further the top floor constructed of such strength that if the roof is not also constructed of fire-resisting material it would successfully withstand the collapse of the roof in case of fire.

Many fires have become a total loss owing to the roof or one floor collapsing and causing the whole series of floors to collapse like a pack of cards unsupported.

Clause 74, Sub-sections 2 and 3, should be amended similar to the suggestion for Clause 68.

Clause 80 with but slight qualifications should be made applicable to existing buildings as well as to new buildings.

In the London Building Acts (Amendment) Act, 1905, we have a revised list of materials to be deemed fire-resisting, as follows:—

(I). For general purposes:—

1. Brickwork constructed of good bricks, well burnt, hard and sound, properly bonded, and solidly put together.
 - (a) With good mortar compounded of good lime and sharp clean sand, hard clean broken brick, broken flint, grit or slag, or
 - (b) With good cement, or
 - (c) With cement mixed with sharp clean sand, hard clean broken brick, broken flint, grit or slag.
2. Granite and other stone suitable for building purposes by reason of its stability and durability.
3. Iron, steel and copper.
4. Slate, tiles, brick and terra-cotta when used for coverings or corbels.
5. Flagstones when used for floors over arches, but such flagstones not to be exposed on the under side, and not supported at the ends only.
6. Concrete composed of broken brick, tile stone chippings, ballast, pumice or coke breeze and lime cement or calcined gypsum.
7. Any combination of concrete and steel or iron.

(II). For special purposes:—

1. In the case of doors and shutters and their frames, oak, teak, jarrah, karri, or other hard timber not less than 1½ in. finished thickness, the frames being bedded solid to the walls or partitions.
2. In the case of staircases and landings, oak, teak, jarrah, karri, or other hard timber. The treads, strings, risers and bearers being not less than 1½ in. finished thickness, and the ceilings and soffits, if any, being of plaster or cement.
3. Oak, teak, jarrah, karri, and other hard timber when used for beams, posts, or in combination with iron, the timber and the iron (if any) being protected by plastering or other incombustible or non-conducting external coating not less than 2 in. in thickness, or in the case of timber not less than 1 in. in thickness on iron lathing.
4. (a) In the case of floors and roofs.

Brick, tile, terra-cotta or concrete composed as described in paragraph I. (6) of this schedule, not less than 5 in. thick in combination with iron or steel.

(b) In the case of floors and of the roofs of projecting shops.

Pugging of concrete composed as described in the said paragraph I. (6), not less than 5 in. thick between wood joists, provided a fillet 1 in. square is secured to the sides of the joists, and placed so as to be in a central position in the depth of the concrete, or concrete blocks not less than 5 in. thick, laid between wood joists on fire-resisting bearers secured to the sides of joists.

5. In the case of verandahs, balustrades, and outside landings, the treads, strings and risers of outside stairs, outside steps, porticoes and porches, oak, teak, jarrah and karri, or other hard timber, not less than 1½ in. finished thickness.

6. In the case of internal partitions, enclosing staircases and passages, terra-cotta, brickwork, concrete or other incombustible material not less than 3 in. thick.

7. In the case of glazing for windows, doors, borrowed lights, lantern or skylights, glass not less than ¼ in. in thickness, in direct combination with metal, the melting point of which is not lower than 1,800 degrees Fahr., in squares not exceeding 16 square inches, and in panels not exceeding 2 ft. across either way, the panels to be secured with fire-resisting material in fire-resisting frames of hard wood, not less than 1½ in. finished thickness or of iron.

(III). Any other material from time to time approved by the Council as fire-resisting.

In Section 1, Sub-section 2 of this list, we have "stone suitable for building by reason of its solidity and durability."

Certain stone which meets the requirements of this clause are quite unsuitable for fire-resistance, and might come under the heading of incombustible material but not fire-resisting.

Slates under Clause 1, Sub-section 4, should be omitted for corbels. Slates will not successfully stand heat and water.

In Clause I., Sub-section 6, concrete formed of coke breeze is permitted, but should be prohibited where it would come in contact with steel, and should not be taken as applicable to I., 7.

In II., 3, plaster protection is allowed to be 1 in. in thickness on timber in lieu of 2 in., if metal lathing is used. From certain fires, it would appear that the metal lathing was of very little help in retarding fire.

In II., 4 (a), concrete requires to be not less than 5 in. in thickness. This I suggest for roofs could be safely reduced to 3½ in. and to 4 in. for floors.

In II., 4 (b), pugging, which is generally very indifferently executed, should be omitted from fire-resisting materials.

Clause 7 of the 1905 Amendment Act I should like to see amended to read that the Council is to be satisfied as to means of escape in case of fire to *all* buildings irrespective of whether they are "high buildings" (that is upper storey greater height than 50 ft.), or provide sleeping or working accommodation for more than twenty people.

(In the case of workshops employing 20 or less I believe one of the Government Departments look after.)

Insurance Companies' Requirements.—It might be of interest to slightly touch on the insurance companies' requirements.

STANDARD 1B.	STANDARD 2.	STANDARD 3.
As to fire-resisting construction they have various standards :—		
Such as Standard 1A, Standard 1B, Standard 2, Standard 3.		
The Standard 1A deals with cotton flax, woollen and worsted mills.		
The Standard 1B requires that buildings be not more than 80 ft. in height and cubic contents of any one compartment not to exceed 60,000 cubic ft.		
Walls to be of brick, terra-cotta, or concrete, and not less than 13 in. thick, but if plain concrete not less than 20 in.	Ditto, but not less than 13 in. thick.	Ditto, but not less than 9 in. thick.
Partitions to be of incombustible material, except—	Ditto, but parti-	

STANDARD 1B.

ing only office enclosures, which are to be of hard non-resinous wood.

All flues to have brickwork not less than 9 in. thick towards the interior, and no woodwork to rest in or be plugged into the brickwork of any flue.

Openings in external walls not to exceed half the area of any storey.

All window frames and sashes to be of iron or other hard metal, and all windows above ground floor to be glazed with glass not less than $\frac{1}{4}$ in. thick in sections not larger than 2 ft. sup., and all openings above the ground floor opposite, and within 20 ft. of any window or other glazed opening, and within 20 ft. of any roof to be protected by "Fire-proof" shutters or doors.

Floors to be brick arches, terra-cotta, fire-clay, or concrete not less than 6 in. thick.

Wooden flooring permitted if laid close to floor with no space.

Solid wood floors, not less than 9 in. thick if ceiled with plaster and covered with floor boards (no space), are also allowed provided waterproof lining is provided underneath the floor boards.

Scuppers to carry off water to be provided of 21 square inches every 12 ft. apart on each floor (buildings in C. of L. or L.C.C. scuppers not essential).

Roofs to be entirely of incombustible materials as for floors, but not less than 4 in. thick.

Glass not less than $\frac{1}{4}$ in. thick, in sections, not exceeding 36 square inches, set in hard metal and wired glass and electro copper glazing considered as incombustible

Structural Metalwork. All columns and stanchions to be covered with brickwork or porous terra-cotta 2 in. thick, or concrete or plaster $1\frac{1}{2}$ in. thick, keyed into metal supports and protected for a height of 4 ft. from floor where cement concrete or plaster only used.

All other metal work, including roof work, to be encased in porous terra-cotta 2 in. thick, securely anchored, or cement concrete or plaster 1 in. thick, keyed into metal supports. Provision to be made in all cases for expansion.

STANDARD 2.

tions of metal lathing and plaster on wood frame allowed.

Ditto, but if rendered with 1-in. cement brickwork may be $4\frac{1}{2}$ in.

Ditto.

Ditto, but in lieu of "fireproof" shutters wired glass may be used subject to certain conditions.

Ditto.

Ditto.

Ditto.

Roofs to be entirely of incombustible material, glass allowed in roof if not less than $\frac{1}{4}$ in. thick not exceeding 5 ft. sup., or wired glass or electro copper glazing, in each case set in hard metal outlets to meet requirements of Factory and Workshop Acts allowed subject to certain conditions.

All columns, stanchions, girders, joists, lintels and other metal work (excluding framework of roofs) to be cased 2 in. thick or with 1 in. concrete or plaster as Standard 1B.

STANDARD 3.

Ditto, but 3 in. thick.

Ditto.

Ditto, but 6 in. thick, and all supports and stanchions and other structural metal work covered with plaster or concrete, etc., $\frac{3}{4}$ in. thick.

Having no combustible material in its construction. Flat roofs of similar construction to solid wood floors, but 4 in. thick. No floor boards, but on upper side covered with metal, concrete, asphalt, or a composition of asbestos and other incombustible mineral ingredients or vulcanite having a coating of at least 2 in. of sand or gravel.

See previous note.

STANDARD 1B.	STANDARD 2.	STANDARD 3.
ROOFS AND CEILINGS. No lining of wood or textile fabric allowed to any part of walls, partitions, ceilings or roof.	Ditto, except dado allowed in wood not exceeding 6 ft. high in offices, cellars, staircases and passages provided not used for storage.	
FLOOR OPENINGS. Only holes for driving shafts, iron or earthenware tubes for electric conductors, and these to be specially dealt with. <i>Staircases</i> practically as required for emergency exits by L.C.C. Where staircases and hoists extend to top floor, enclosure must be roofed with glass roof protected (subject to certain restrictions in L.C.C. area). All belting and rope races to be enclosed as for staircases and hoists. <i>Shafting</i> through walls to fit closely into wall, and to have closed wall boxes leaving no open space. <i>Pipes, etc.</i> No wooden casing allowed. All pipes (except water pipes not exceeding 1½ in. diameter) to be of hard metal.	Ditto. Ditto. Ditto. Ditto. Ditto.	Ditto. Hoists to be enclosed in walls 6 in. concrete or R.C. 3 in. thick with fire-resisting doors. Sprouts or trunks not exceeding 4 in. square feet in area constructed of iron ¼ in. thick and shutters to every opening. Openings up to 12 square feet with double iron or metal covered trap doors (special construction). Staircases having steps and landings of brick, stone, iron, concrete enclosed with walls, this applying also to vent shafts, all openings being protected.
COMMUNICATING COMPARTMENTS. Two or more compartments (constructed according to the rules) may communicate provided their aggregate cubical contents do not exceed 60,000 cubic feet. When cubic contents exceed 60,000 cubic feet communication only allowed across a fireproof compartment, built up from the basement with walls of solid brickwork, and having all openings protected by fireproof doors at least 6 ft. apart.		
REINFORCED CONCRETE. Reinforced concrete buildings allowed subject to the usual precautions. The requirements as to proportion of cement in the concrete are somewhat ambiguous, viz. :— “In the proportion of six cwts. of cement to each cubic yard of concrete.” All external walls to be not less than 6 in. thick.	Ditto Ditto. Ditto.	Ditto. Ditto. Between piers 5 in., ditto.

STANDARD 1B.	STANDARD 2.	STANDARD 3.
Division walls 8 in. thick.	Ditto.	
Party walls 13 in. thick except if adjoining reinforced concrete building, then 8 in. thick.		
Floors 4 in. thick if lined with fire tiles 1½ in. thick.	Ditto.	
Floors 5 in. thick and supported on beams and columns of reinforced concrete.	Ditto.	Ditto, but 3 in. thick.
Roofs 3 in. thick	Ditto.	Ditto.
No metal to be nearer face than double its diameter, but not less than 1 in. and need not exceed 2 in.	Ditto.	Ditto.
Enclosures and staircase and hoists 6 in. thick.		Ditto.
Fire-resisting compartments to have walls 8 in. and floors 5 in. thick.		Ditto.

Under Standard IV., the party and external walls to be not less than 9 in. thick of brickwork, etc., and no combustible material to enter into any part of construction except the roofing, doors and window frames.

External walls of reinforced concrete 5 in. thick.

Hoists enclosure to be of incombustible construction.

Staircases may have steps and landings of hard wood.

The insurance companies' regulations as to fireproof doors and shutters are many and various, they deal with iron and steel (sliding and hinged). Metal-covered doors (ditto). "Check fire" doors (ditto). Ferro-concrete doors, etc.

The openings must not exceed 56 ft. sup., and not more than 7 ft. in width or 9 ft. in height.

In some cases not to exceed 45 ft. sup., and not more than 6 ft. in width or 8 ft. in height.

In other cases 35 ft. sup., and not more than 5 ft. in width or 7 ft. in height.

These rules generally are very careful as to fixing and little details all of the greatest importance.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY—NEW BUILDINGS.

IN an article in a recent issue of the *Technical Journal of Association of Teachers* it is stated that "Up to the period of the Civil War technological education in the United States was represented only by the United States Military and Naval Academies in military engineering, by the Rensselaer Polytechnic Institute and Union College in Civil Engineering, and by the recently established scientific departments of Harvard and Yale Colleges.

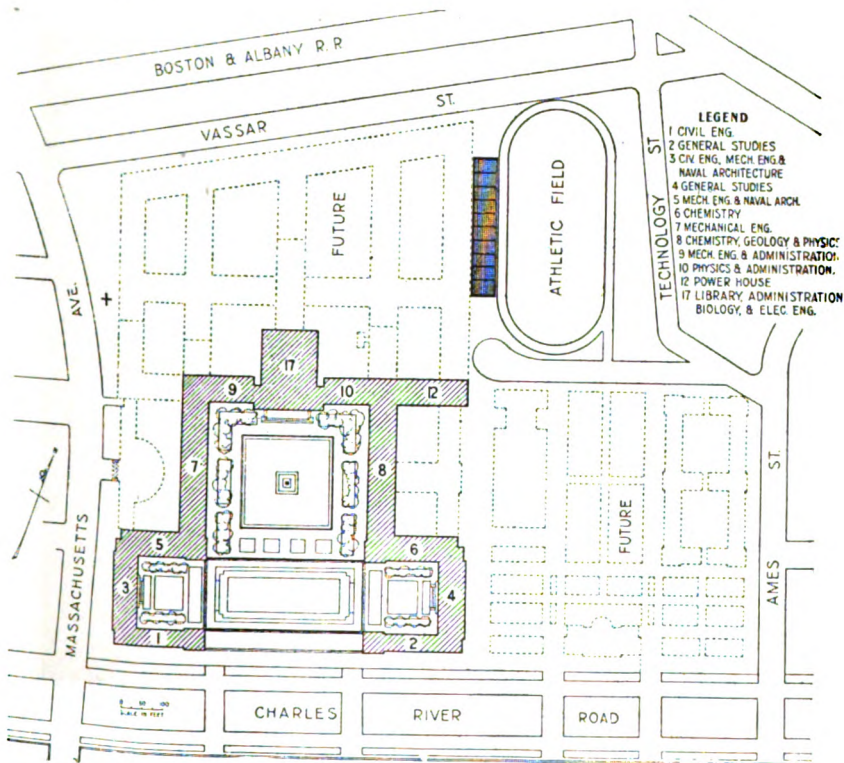


Fig. 1. Buildings under construction, shaded lines. Future construction, dotted lines.
NEW BUILDINGS—THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

"On April 10th, 1861, on the very eve of the Civil War, an Act was passed by the State Legislature to incorporate the Massachusetts Institute of Technology 'for the purpose of instituting and maintaining a society of arts, a museum of arts, and a school

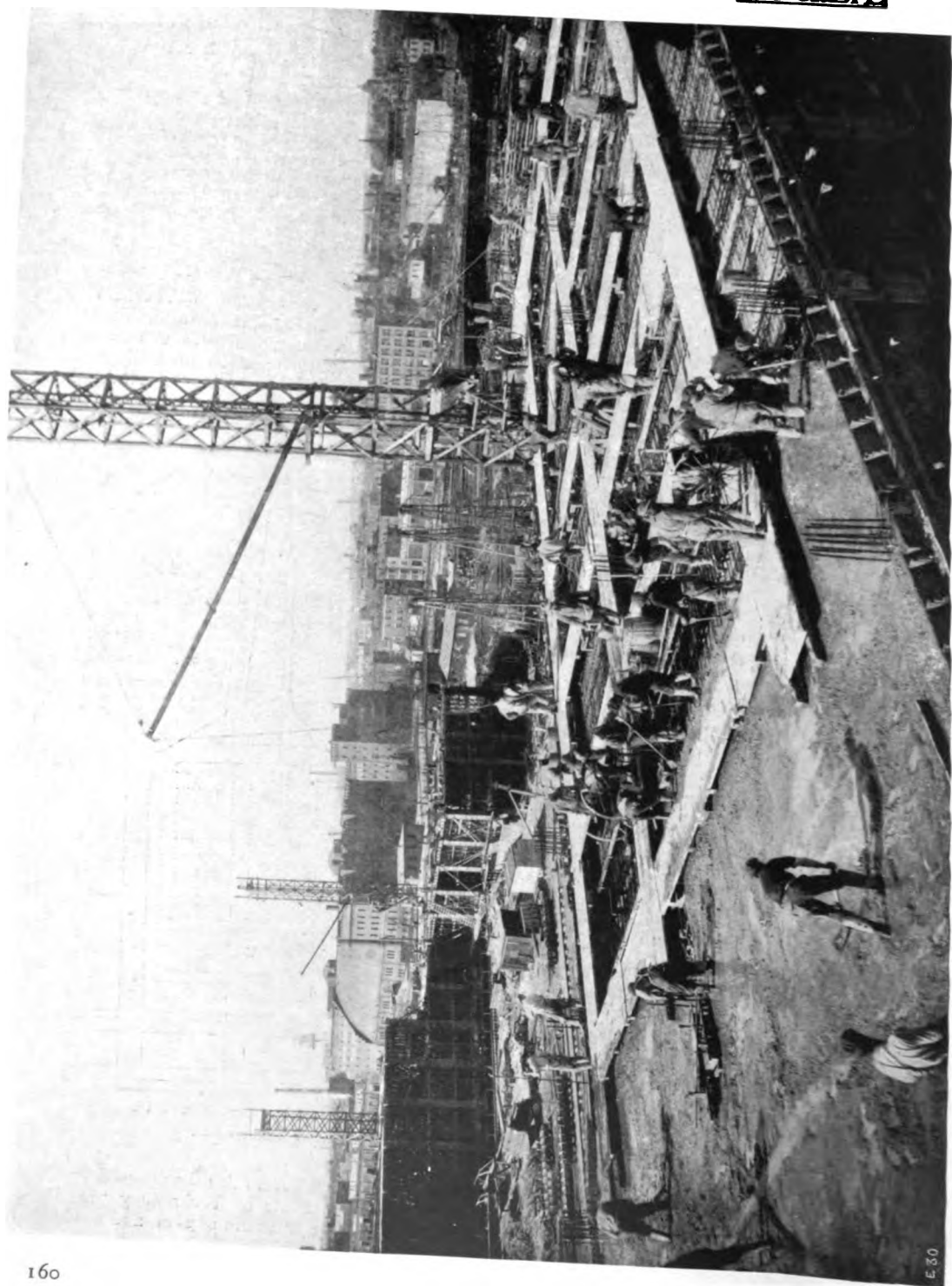


Fig. 1. Concrete Pouring on 3rd Floor of Building No. 8.

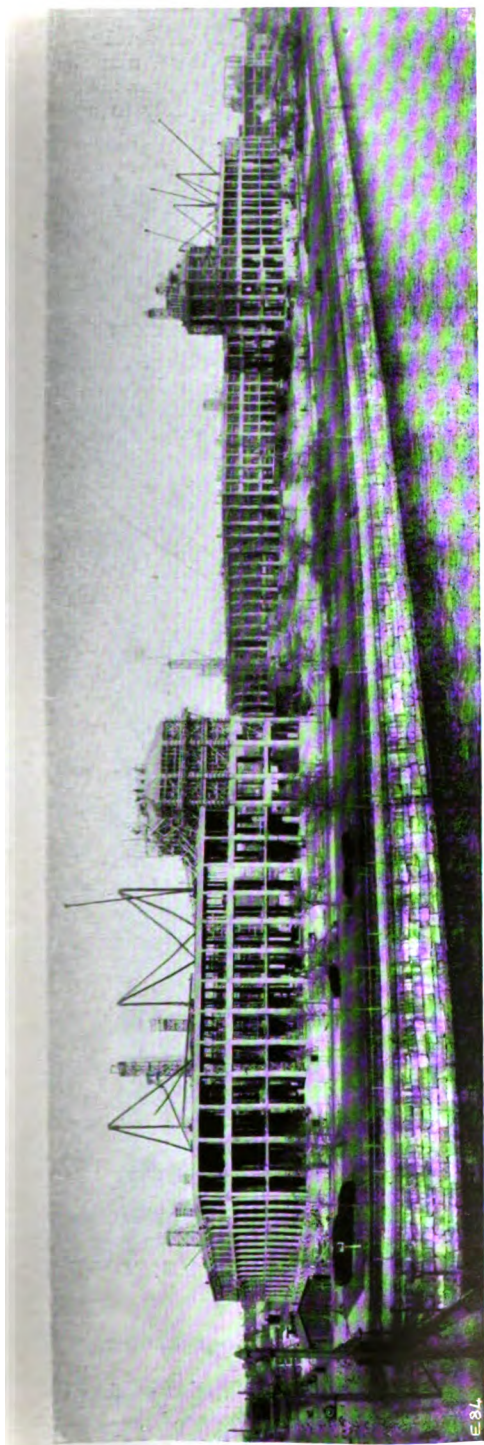


Fig. 3. State of Buildings at the beginning of November, 1914.

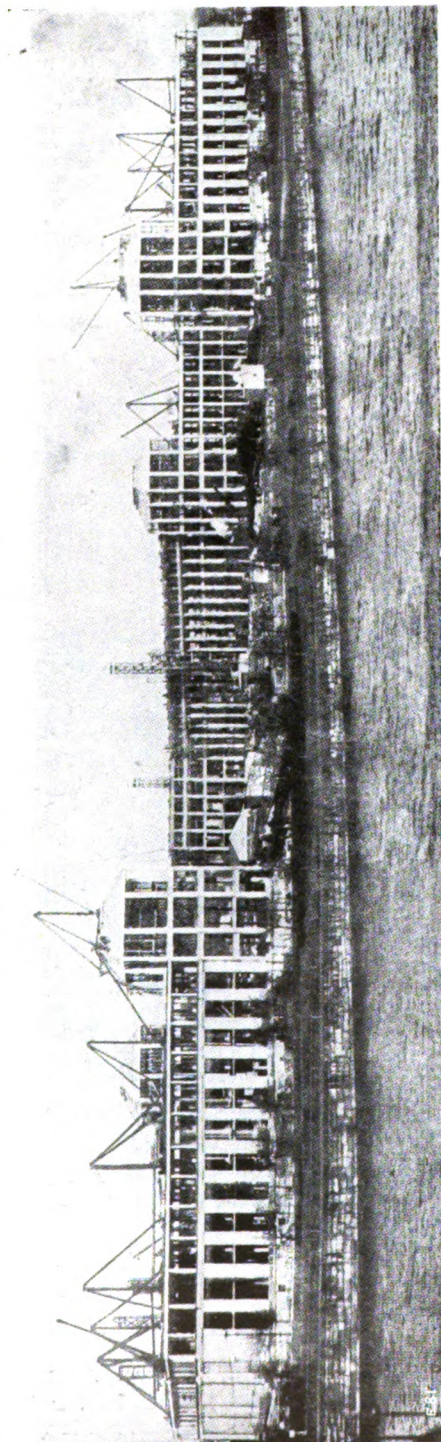


Fig. 4. View of New Buildings from Harvard Bridge.
NEW BUILDINGS—THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

of industrial science, and aiding generally by suitable means of advancement, development, and practical application of science in connection with arts, agriculture, manufacture and commerce.' A small tract of newly-filled State land in Boston was set apart for the use of the new Institute, which was authorised to hold property to an amount not exceeding \$200,000."



Fig. 5. Showing Pouring of Floor Concrete.
NEW BUILDINGS—THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

The Institute originally started with a single building, but it has been necessary from time to time to expand it in various directions.

Recently a further tract of land was bought of about 50 acres on the Cambridge side of Charles River separating Boston from Cambridge.

It is with the buildings now in course of erection on this site that we are concerned

in the present notes. It is stated that the land, with the buildings now being erected upon it, will cost about five million dollars.

The buildings consist entirely of lecture-rooms, laboratories, and the like, but it is expected that later on dormitories will be erected and other provision made for the social life of the institution.

It was to be expected that in a large building undertaking of this kind reinforced concrete would certainly enter into the construction.

Fig. 1 gives the general lay-out showing the buildings now under construction and also indicates future work to be undertaken.

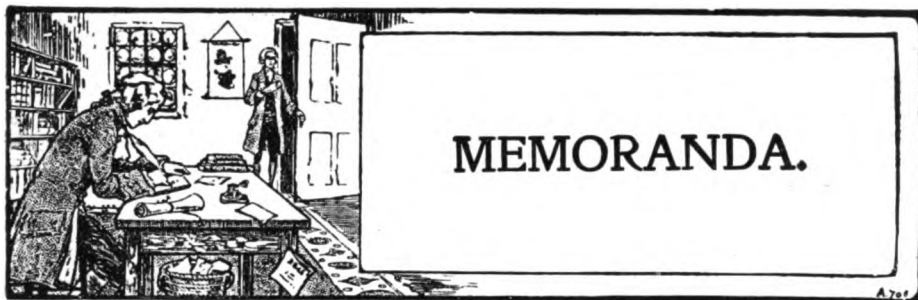
The plans for the new buildings were approved in July of last year and since that date work has advanced rapidly.

By about October the excavation work was completed (a total of 65,000 yds.), and, with the exception of the library, pile driving had been finished, some 20,000 piles having been driven. Further, about 15,000 cu. yds. of concrete had been poured, 600,000 sq. ft. of forms had been erected, and 1,000 tons of reinforcing steel placed.

Considerable progress had been made on the work between October and the beginning of the current year, when practically three-fourths of the concrete work for all the buildings, including the library, was completed.

The accompanying illustrations show the work at different stages of construction and will give some idea of the extent of the scheme. We hope to be able to supplement the present preliminary notes by a fuller article at a later date.

The work is being carried out by the Stone and Webster Engineering Corporation, of Boston, to whom we are indebted for our illustrations and some of our particulars.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The Institution of Civil Engineers.—At a meeting of the Institution last month Mr. F. D. Evans, A.M.Inst.C.E., read a paper, entitled "Engineering Operations for the Prevention of Malaria," and we give the following short extract from the paper:—

Anything which relates to the health of communities and labour forces is of importance to engineers, but the disease referred to in this paper, malaria, is of particular interest to them. The effects of malaria are often terrible, and may lead to very high death-rates. Its effect on works in new countries is to greatly increase their first cost and cost of maintenance, and it may lead to their abandonment.

Malaria occurs more or less in all warm climates, and is caused by parasites of the blood which are injected by anopheline mosquitoes. These mosquitoes first take the parasite from infected man when sucking blood. Thus part of the life of the parasite is spent in man and part in the mosquito. Quinine is the remedy for malaria. No attempts to transmit malaria except through the agency of mosquitoes have been successful.

The most important method of reducing anophelines is by drying land by means of thorough drainage. In hill-land subject to heavy rainfall such work presents many novel problems, but has been successfully accomplished in the Federated Malay States at a reasonable cost.

Ravines in the Malay States, unless exceptional, can only be laid dry by a continuous drain running completely round the hill-foot, cut on a line at a little above the wet surface level. Open earth or masonry drains are useless for the purpose, and it has been found that the only way to construct permanent hill-foot drains is to use agricultural drain-pipes as subsoil drains, and to provide outlet drains of open masonry or subsoil pipes to carry off their discharge. The pipes are proportioned to run not more than one-quarter full in ordinary dry weather (one-fifth full for drains over 6 in. in diameter). The use of pipe drains less than 4 in. in diameter is not ordinarily advisable, owing to the liability of their choking. The pipes are laid at a depth of 3 ft. to 4 ft. (or even deeper on steep slopes); they have been laid at the heads of ravines on slopes of 1 on 3 with satisfactory results.

Trees must be removed from the vicinity of pipe drains, as their roots may grow into and choke the pipes. Undergrowth and long grass should not be allowed to grow up, for the same reason.

As economy is all-important in anti-malarial work, it is rarely possible to provide masonry drains large enough to carry all flood-water, and these are, therefore, usually calculated to run one-sixth to one-fifth full in ordinary dry weather, and are surcharged during floods. An inexpensive and thoroughly efficient type of drain has been evolved to meet the conditions, formed of concrete blocks of half-egg shape, laid close but unjointed. The blocks are laid without foundations even on bad ground in flowing water. Should they move out of line or gradient, it is easy to reset them correctly when the surrounding ground has settled, after which they give no trouble; but resetting is rarely necessary.

Iron and Steel Institute.—The annual meeting of the Institute will be held, by kind permission, at the Institution of Civil Engineers, Great George Street, London,

S.W., on Thursday and Friday, May 13th and 14th. Thursday, May 13th, has been fixed provisionally as the date for the annual dinner.

It has been decided for the present to hold the autumn meeting in London during the week ending September 25th.

The King George Hospital.—We note from the daily press that H.M. Stationery Office, regarding which two articles appeared in our Journal, is to be turned into a hospital for the wounded soldiers from the front.

As can well be understood, a good deal of work is necessary to convert this large and extensive building, originally intended for the Stationery Department, into suitable quarters for hospital work. The following are a few particulars regarding this hospital:—

On the fifth floor the special feature will be a long row of kitchens, whence the food will be sent by lifts in heat-preserving receptacles to the ward kitchens, of which each lower floor will have seven. On this floor are also two dental rooms, and there will also be a number of beds.

The second, third, and fourth floors will be given over to beds, mostly for surgical cases. Each of these floors will have two operating theatres.

On the ground floor will be the administrative offices, barrack room, a recreation room, dispensaries, and laboratories.

In the basement there is to be a well-equipped department for X-ray apparatus, and there will also be storage room.

The flat roof, too, will be turned to account, for it is to serve as an open-air recreation ground when conditions permit.

Altogether this vast building will have 63 wards, 1,650 beds for patients, and six operating theatres.

The Improvement of the River Trent.—With the object of improving the navigable facilities of the River Trent below Nottingham a scheme which will involve the expenditure of about £160,000 has recently been decided upon. The scheme comprises briefly the deepening of the river between Nottingham and Newark and the provision of locks in order to make the former city an inland port accessible for larger boats than can at present use it between this place and the River Humber.

The necessary powers were obtained in 1906 to construct additional locks and weirs beyond those already carried out under former powers obtained. Under the powers of 1906 the Cromwell lock was built. It is 188 ft. long between the gates and 30 ft. wide. These dimensions were adopted in order to provide accommodation at one time for four of the present standard Trent boats or one tug and three boats, as is customary in the navigation of the river. The sills of the gates are placed at such levels as to ensure in any dry period a minimum depth of water of 6 ft., and the walls and gates are carried to a sufficient height to enable the lock to be navigated in flood time. A weir has been provided with shutters.

Practically the whole of the work was carried out in Portland cement concrete, the ballast for which was obtained from the excavation for the lock.

The walls have a maximum height of about 30 ft. from the bottom of the foundation to the coping level, and were built in heavily timbered trenches.

The present proposal comprises numerous new works, including additional locks and weirs, and also a large goods depot on the canal side in Nottingham.

The new works will be carried out by Mr. F. Rayner, A.M.Inst.C.E., the engineer to the Trent Navigation Company, in conjunction with the City Engineer of Nottingham.—Taken from an article in the *Engineer*.

Loughor Bridge.—At a recent meeting of the Carmarthenshire County Council it was reported that the Loughor Bridge Joint Committee have decided to erect a new bridge at Loughor in reinforced concrete. The bridge is to be 670 ft. long between abutments and 36 ft. wide between the parapets, having a 24-ft. carriageway and two 6-ft. footways. There are to be seven intermediate piers, the foundations for which are to be carried on piles in the river bed, the piers themselves being formed of reinforced concrete. The main structure is to be carried from pier to pier by means of six ribs and the slab is turned up at the sides to form a parapet.

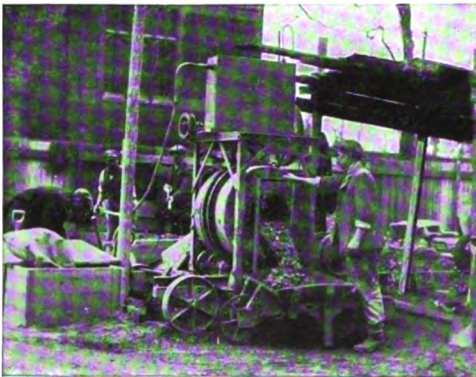
The tender which was accepted was that of the Midland Counties Reinforced Concrete Company, of Birmingham.

PUBLICATIONS RECEIVED.

Reinforced Concrete Construction.—Messrs. F. A. Macdonald and Partners, reinforced concrete engineers, have recently issued a most useful handbook dealing with their system of reinforced concrete construction. In addition to setting out in a very clear and concise manner the advantages claimed for their system, the handbook also contains much other useful information. Some general notes are given regarding the properties of reinforced concrete, notes as to bending moments, and also calculations for beams, columns, and foundation slabs. A general specification for the practical execution of reinforced concrete work is also given. The book is well got up and contains a large number of very good illustrations with short descriptions of works carried out on the Macdonald system. The book will be forwarded to engineers and architects upon application to Messrs. F. A. Macdonald and Partners, 135, Wellington Street, Glasgow.

Lockwood's Price Book, 1915.—This book is so well known among architects, surveyors, and others that it is almost needless to emphasise its utility. In the preface special attention is drawn to the greatly increased price of wood goods on account of the decrease of wood imported, due to the war. Further, wages have increased, and the section dealing with the price of labour has been brought up to date as far as possible.

Every kind of building work is dealt with. There are a number of appendices containing tables of weights and measures; particulars regarding solicitors' costs for conveyances, leases, etc., and stamp duties; valuation tables, wages tables; form of agreement for building contracts sanctioned by the R.I.B.A. Further, the London Building Acts and the various Amendment Acts, 1894-1909, are given in a supplement, including the L.C.C. (General Powers) Act, 1905, prescribing regulations and requirements for steel frame and reinforced concrete buildings.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



View of General Hospital and New Wing.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.
(For Description see p. 119).

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X. No. 4.

LONDON, APRIL, 1915.

EDITORIAL NOTES.

ECONOMY IN REINFORCED CONCRETE CONSTRUCTION.

ELSEWHERE in this issue we publish an extract from a paper which was read before the Concrete Institute by Mr. T. A. Watson, together with some notes of the discussion that followed. There are many who are quite assured that a real and substantial economy can be effected with the use of reinforced concrete, and the great increase in popularity during recent years goes to prove that the material is capable of competing successfully with steelwork in price, speed of construction and adaptability.

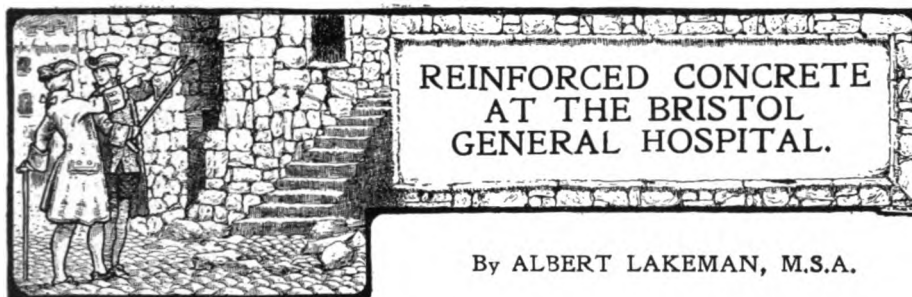
Again, there are many persons connected with the building industry who are just as convinced that reinforced concrete is considerably over-rated, and they will produce estimates which indicate the absence of any saving with the material. These persons are quite honest in their convictions, and not biassed by any financial interest in material or firm. This may seem strange and unaccountable to anyone who has no knowledge of the methods which obtain in the building industry, but when all the facts and conditions are analysed it is not so surprising, and is only indicative of the lack of a definite relation and method of business between client, architect, specialist and contractor. There are many architects who claim to have considered the question of cost by actual tenders for the two materials, and yet if the facts of the matter were known it would be found that no real comparison could be drawn, as the same basis had not been adopted when obtaining the tenders, and the architect, often not really wishing to use reinforced concrete, will put conditions and difficulties in the way of economical design which he does not consider necessary in the case of steelwork. A certain architect was heard to state a few days ago that personally he considered a factor of safety of 10 was necessary in reinforced concrete work, as it was impossible to guarantee the small amount of steel being placed in exactly the right positions and therefore the material was unreliable! With steelwork he was satisfied that it was impossible for any errors to be overlooked, as the work was more open to inspection for a fairly long period of time. This gentleman also stated that he had always found steelwork construction as economical in first cost as reinforced concrete. We should quite imagine that anyone requiring a factor of safety of 10 in the case of the latter material as against a factor of 4 in the former would find steelwork much cheaper. It is obviously quite impossible to compare the relative cost of the two materials when the tenders are obtained in a casual sort of a manner without due regard to economical and efficient design in both cases and without imposing suitable conditions for each scheme which are fair to the material. Generally speaking, it is the architect who will settle the construction to be employed, leaving out for the moment those

schemes which are executed by the contractor direct for the client, and it is necessary that this fact should be realised at the outset, because it has an important bearing on the subject.

Now, if architects as a body could be convinced that an actual saving is effected by the use of reinforced concrete, there is no doubt that structural steelwork would at once take second place in all important work, but there are difficulties to be overcome which must not be overlooked. In the first instance, architects are familiar with steelwork, and can generally calculate the required strength of ordinary members with the aid of available tables, and they have a certain feeling of confidence in dealing with a material which they understand. With reinforced concrete the circumstances are different, there being extremely few architects who thoroughly understand the theoretical side of this material, and there are no tables or "rule-of-thumb" methods for making even approximate calculations; and as they cannot be expected to take up the subject in the same manner as the engineer, and become proficient in the design, they are apt to avoid the material as much as possible.

Secondly, the architect is not limited by any patent system if he decides to use steelwork. But with reinforced concrete he is limited to a certain extent by systems, by specialists who wish to take off their own quantities, by licensed contractors, and by other questions which are not helpful. If he selects a particular firm of specialists he eliminates competition, which he must have for the satisfaction of his client; and the only alternative is to employ a consulting engineer, who will prepare a scheme upon which to obtain estimates. This method is by far the most satisfactory, but it is not always possible to persuade the client to pay the necessary fees to the engineer, and the architect cannot afford to pay them himself. We quite agree with Mr. Watson's remarks as to the great waste which occurs when several firms are asked to prepare calculations and schemes for the same building when only one set of drawings will be used. If it were only possible to accomplish, it would pay all the specialists to combine, in the case of competition schemes, and jointly employ one independent engineer to prepare a set of drawings upon which each firm could tender. The result would be lower prices and a proportionate increase in the use of reinforced concrete, which would eventually be beneficial to all the firms.

We feel that these questions of the relation between the architect and the specialist, and the methods of obtaining schemes and prices, are far more important when considering the relative economy of reinforced concrete than any other points mentioned in Mr. Watson's paper, and if the material is to become universal some satisfactory methods must be devised to deal with these. Reinforced concrete is very adaptable, but it will not always prove cheaper than structural steel, although some enthusiasts hold this opinion. The greatest economy will always be effected where a large amount of repetition in the members is possible and where the conditions allow them to be designed with the economical percentage of steel. If reinforced concrete is designed by an engineer who is conversant with economical and good design, there is no doubt that a real economy will be effected by the use of this material as compared with ordinary structural steelwork, provided the circumstances of the case permit the designer to take full advantage of the special properties of reinforced concrete.



The following description of the Extension Works at the Bristol General Hospital is one of the examples in building work where concrete has been used throughout, inasmuch as reinforced concrete has been used for the constructional members and concrete blocks form the external walls.—ED.

THE extensions that have just been carried out at the Bristol General Hospital comprise the erection of a new wing approximately 176 ft. long and about 40 ft. wide, and reinforced concrete has been employed for all the constructional members. The position of the new wing is a very good one, and the wards are placed axially north and south, so as to receive sun all day. There are five floors in all—viz., basement, ground, first, second and third, and the basement floor is only partly below the ground surface level.

The accommodation in the basement consists of a large dental department, with surgeries, waiting rooms, conversation room, workroom and offices—and dining-room accommodation for the female staff. A new laundry is also provided on this floor with all the necessary departments and appliances.

The ground floor is devoted to six suites of private sitting-rooms and bedrooms for resident officers, smoking and common rooms, messrooms, registrar's room and sundry offices.

On the first floor the space is allocated to a women's medical 25-bed unit, comprising a ward of 22 beds with sun balcony at the south end, one single-bed ward and one two-bed ward, sister's room, ward kitchen, larder, and the necessary rooms for storing linen and patients' clothes.

A sanitary wing is also constructed as a small block off the wing building. On this floor there is also a clinical laboratory.

The second floor is designed so that it can be converted to provide exactly the same rooms as mentioned for the first floor, but at present it is arranged as a maternity ward, with a large open-air balcony at the south end. A part of the building is roofed in at the third floor level with a flat roof that can be utilised as a garden or open-air exercise space, and the remainder of the third floor is given up to sleeping accommodation for the nurses. Twenty-two additional bedrooms have been provided, some of which were obtained by an extension of the top floor of the existing building adjacent to the new wing.

The whole of the constructional members are of reinforced concrete on the Hennebique system from the detail drawings of Messrs. Mouchel and Partners, 38, Victoria Street, Westminster, and the building is a complete concrete structure, as the whole of the external walls are built of concrete blocks made on "Winget" machines.

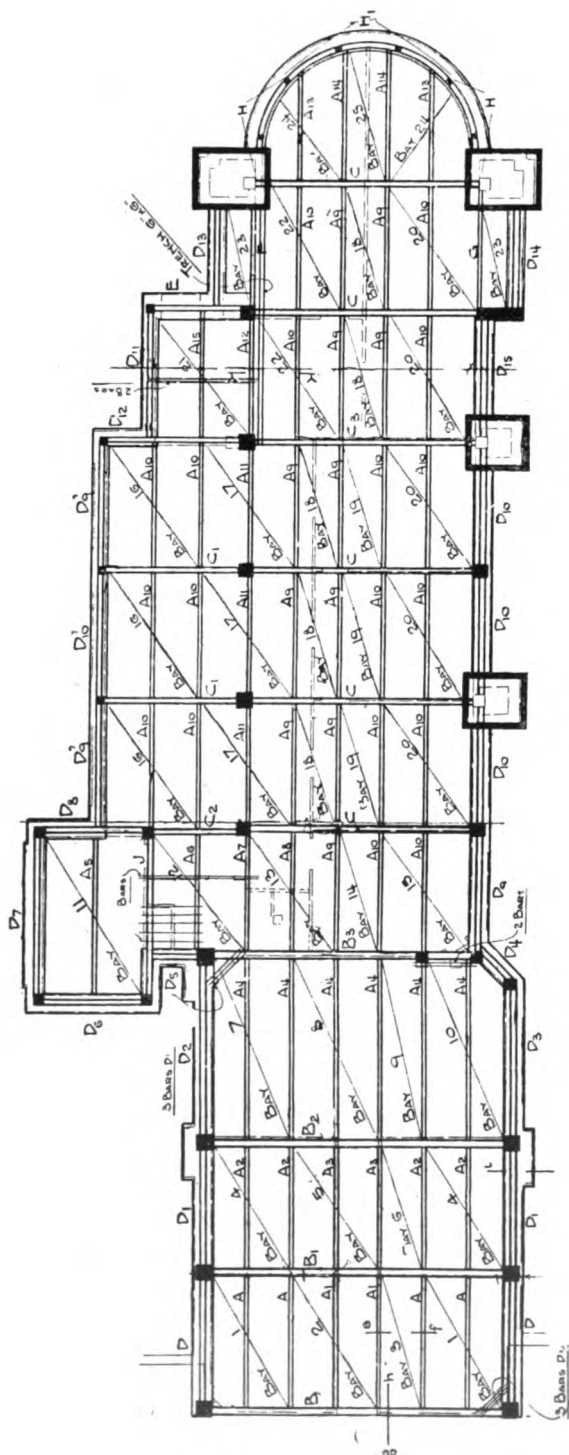


Fig. 1. General Plan of Basement Floor.

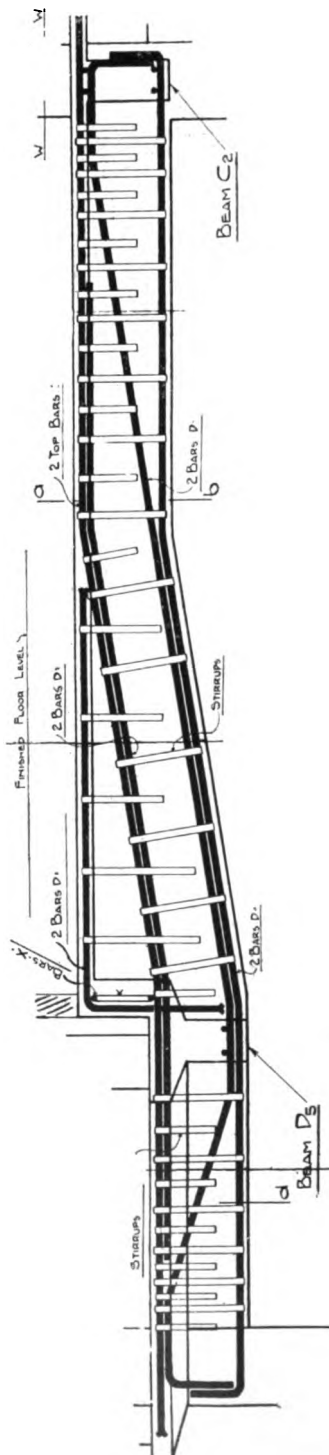


Fig. 2. Section showing Elevation of one of the Basement Floor Beams. REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.

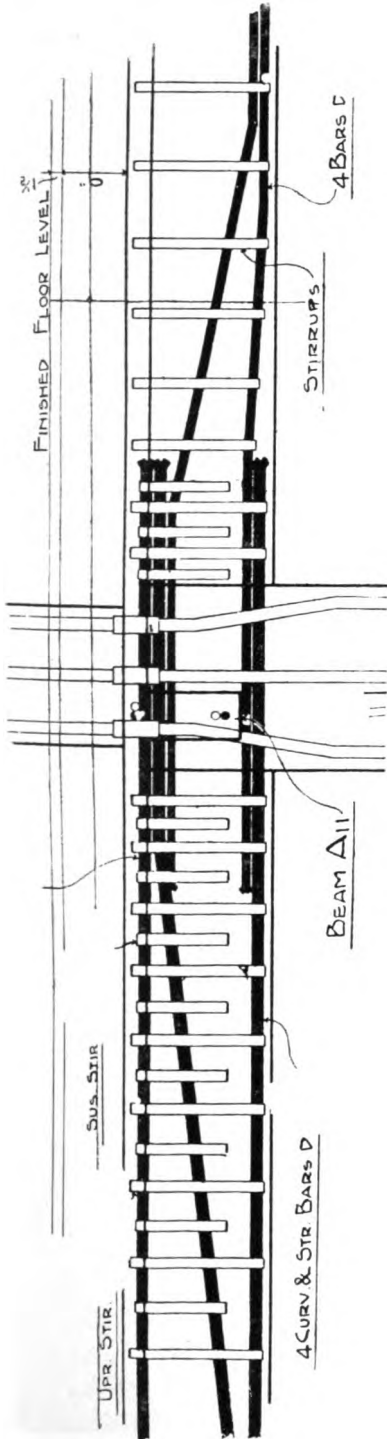


Fig. 3. Elevation of Column, Basement Floor.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.

Columns.—The columns are arranged generally in two longitudinal rows, and they occur in the outer walls, thus leaving the whole of the floor space clear and available for any internal division. These two rows are about 38 ft. apart at that end of the wing adjacent to the old building and about 29 ft. apart for the remainder of the length. In the basement a row of secondary columns is formed about 18 ft. from one of the main rows on the outside of it, owing to the width of the basement floor being greater, for a portion of the length, than the floors above. The spacing of the columns is about 16 ft., centre to centre, and at the south end they are arranged on the circumference of a semicircle at about 7 ft. centres to carry the curved balconies which are provided at the end of the building. The columns vary in size according to position and load, and the largest section employed is 24 in. by 24 in. with eight lines of vertical reinforcement tied with links at 4-in. centres. The load on the soil was taken at 4 tons per sq. foot, and the reinforced concrete foundations to the columns were carried down some distance below the basement floor level on account of the nature of the soil. The largest bases are about 8 ft. sq., and these are about 21 ft. below the basement floor, which is slightly less than the maximum depth of 24 ft. employed in some cases.

Beams and Floors.—A plan of the basement floor, showing the general lay-out of the beams, is illustrated in Fig. 1, where it will be seen that the main floor beams span the width of the building and are carried by the two longitudinal rows of columns above mentioned, while secondary beams are introduced between at about 5 ft. 6 in. centres. The main

beams are of two types, viz., those having a span of about 38 ft. and those across the narrower portion of the building, which have a span of about 28 ft. The first mentioned have a total depth of 27 in. and a width of 12 in., with six bars in the lower surface and three in the upper at the centre, one half of those in the lower surface being bent up towards the top at the ends. Vertical stirrups are provided to both sets of rods throughout the length of the beam. One of these beams, which supports a larger floor area than the others, has six bars on both the upper and lower surfaces. The second type of main beam has a depth of 19 in. and a width of 8 in., with four bars in the lower surface and two in the upper. A detail showing the connection between two of these main beams and a column is illustrated in Fig. 3, where the arrangement of the bars can be clearly seen. The secondary beams are 16 in. deep and 6 in. wide, reinforced with two bars in the lower surface, and the floor slab is 3 in. thick only, with rods in both upper and lower surfaces. The type of floor here employed does not follow that used in the other floors, as will be seen later. A somewhat curious beam is that illustrated in Fig. 2, this having a span of about 20 ft. and occurring at the junction of the sanitary block and the main building. The bearing at either end is at different levels, and it is, therefore, cranked in its length with a horizontal top surface and a partly horizontal and partly sloping soffit. Owing to the expense of carrying down the walls to the required depth to obtain a satisfactory bearing, as in the case of the columns, these are carried at a level just below the basement floor by beams between the columns, as shown in the detail illustrated in Fig. 4. These beams generally are designed as tee beams with a total depth of 19 in. and a 9-in. rib and 30-in. flange with four bars as reinforcement on the lower surface. In some cases it was necessary to project the portion of the flange on one side of the rib for a greater distance owing to projection beyond the main face of the wall, and such portion had to be calculated as a cantilever slab. The tailing down is provided by the beam taking the edge of the floor which runs parallel to the wall coincident with the inner face of the columns.

The construction for the upper floors is somewhat different to that just described, as it consists of main beams transversely across the building between the columns, and the bays so formed are filled in with a light floor composed of reinforced concrete and hollow concrete tubes on the Hennebique system. The main beams on the widest portion of the building are 23½ in. deep and 16 in. wide with eight rods in the lower surface, and in the narrower portion they are 23½ in. deep and 12 in. wide with four bars in the lower surface, and in some cases two bars are provided on the upper surface in addition. A portion of the ground floor at one side is roofed in with a sloping roof and a detail of the sloping beams, the main beam in the adjoining bay and the connection with the

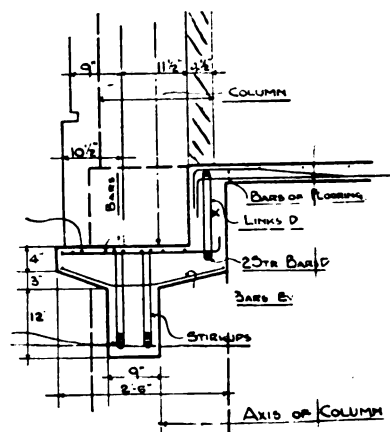


Fig. 4. Detail of Lintel Beam.
REINFORCED CONCRETE AT THE BRISTOL
GENERAL HOSPITAL.

column is shown in Fig. 6. The sloping beam is 18 in. deep, 24 in. wide across the flange at the top, and 12 in. wide across the rib, and it is reinforced with eight rods, four in each surface, for the greater part of its length. The floor panel construction is of three types to suit the different sizes of the bays.

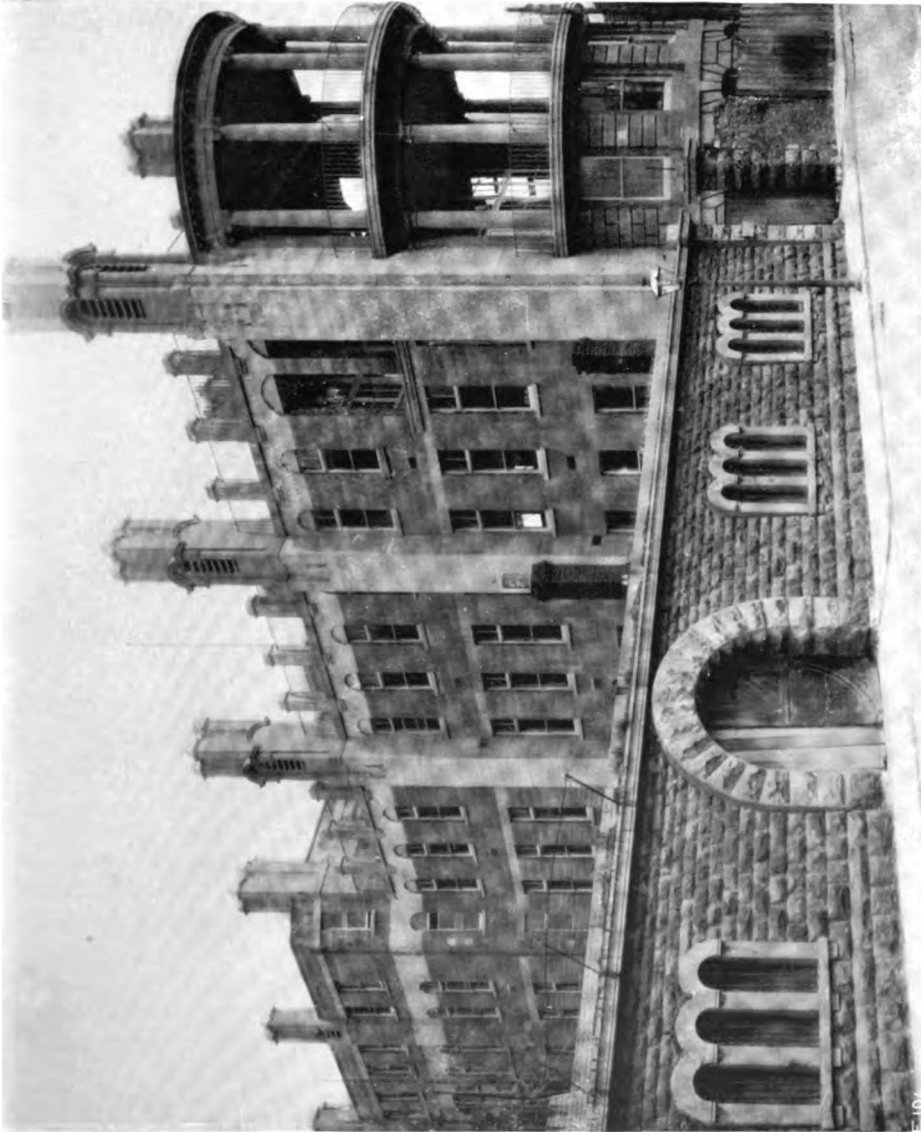


Fig. 5. New Wing, Finished Building.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.

The smallest type has a constructional thickness of 8 in. with tubes at 18 in. centres and a minimum thickness of $1\frac{1}{2}$ in. of concrete over the tubes. The other types are 12 and 14 in. deep respectively, with tubes at 24-in. and 28-in. centres. The tubes are placed 3 in. apart on the clear at the top, and with the sloping sides this gives a thickness of 4 in. of concrete at the bottom, and the concrete

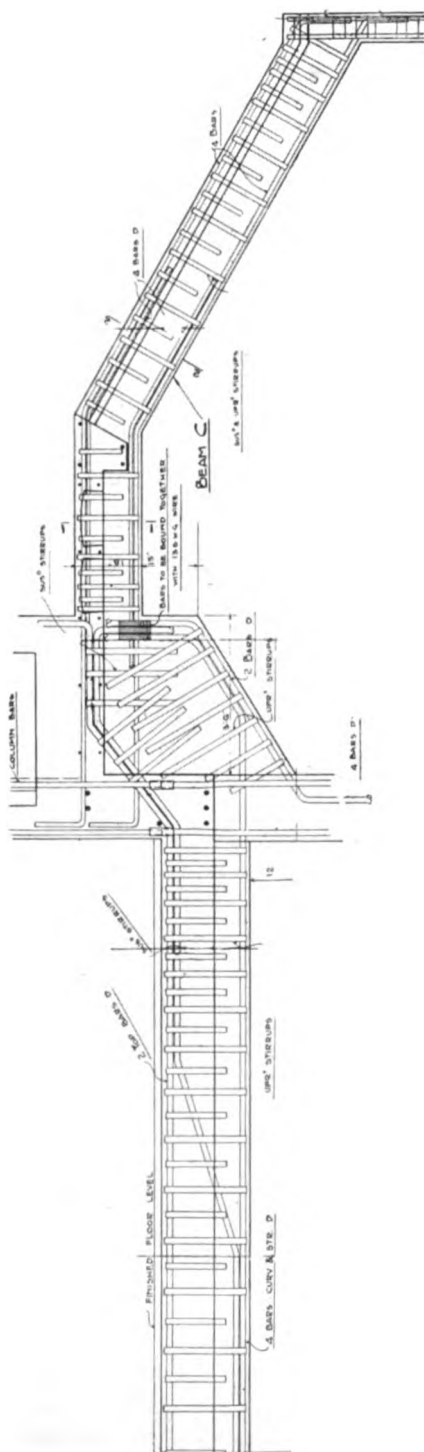


Fig. 6. Cranked Beam, Ground Floor.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.

is reinforced with two bars on the lower surface at the centre of the span. One of these bars is cranked up towards the top and the other is continued right through in the lower surface. Bars are also placed in the top surface over the tubes and at right angles to them. These floors are very light for their strength, and the shuttering required is reduced to a minimum as the tubes act as a permanent centering to the greater part of the concrete.

External Walls.—The external walls are carried by lintel beams between the columns at the floor level, these walls being built with an outer leaf of 9 in., constructed of hollow concrete blocks, a 3-in. cavity, and an inner leaf of brickwork $4\frac{1}{2}$ in. thick. The exterior treatment of the building with the concrete blocks is very satisfactory, as will be seen in the photographic views in Fig. 5, while the construction is light and durable. The shafts that can be seen in this view, and which extend above the roof level, are used as extraction shafts in connection with the ventilation. They are formed in reinforced concrete and generally have a thickness of 6 in. with bars in both surfaces and in both directions. The interior is readily accessible and can be hosed down when required. The open air balcony which occurs at the south end of the wing is carried at each floor level by curved lintels 16 in. deep and 9 in. wide, reinforced with two bars in both upper and lower surfaces.

Laundry.—An interior view of the laundry is given in Fig. 9, this being a one storey building with a height of about 12 ft. The foundations are carried down about 6 ft. below the floor level and the walls are carried on reinforced concrete slabs

8 in. thick and 2 ft. or 2 ft. 6 in. wide, these being reinforced in both surfaces. The columns are 9 in. and 12 in. square with four lines of vertical reinforcement, and the larger type has a foundation slab 5 ft. square with a minimum thickness of 8 in. at the outer edges, increased to 20 in. at the point of intersection with

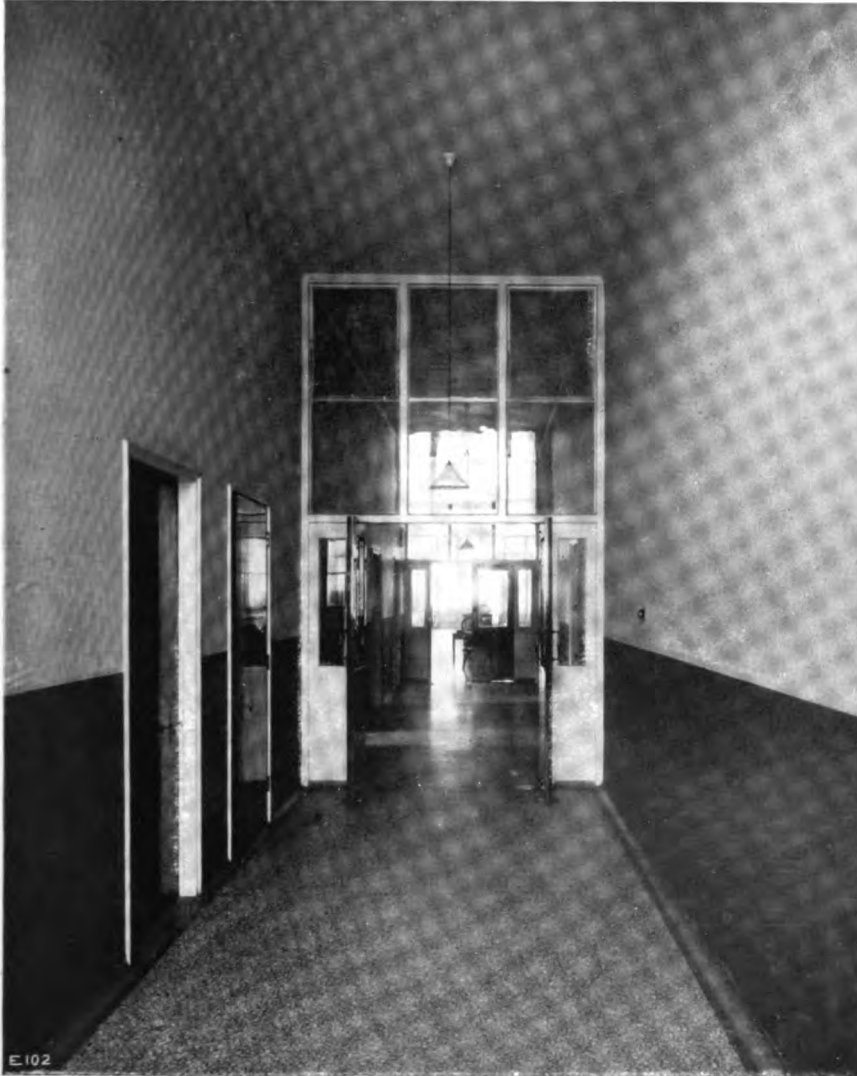


Fig. 7. View of one of the Corridors, New Wing.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.

the column. A lattice of rods is placed on the bottom surface and at the centre of the thickness with the main column rods turned out at right angles to the length, 2 in. above the bottom reinforcement. The roof is constructed with arched reinforced slabs springing from 16 in. by 9 in. reinforced concrete beams, with a central skylight running through each bay, and tie beams are provided

across the bays at intervals of 7 ft. 6 in., these being 18 in. deep and 7 in. wide. These beams come above the slabs between the skylights, and holes are formed through them for the passage of water in the gutters. The whole of the roof and beams are covered with asphalt.

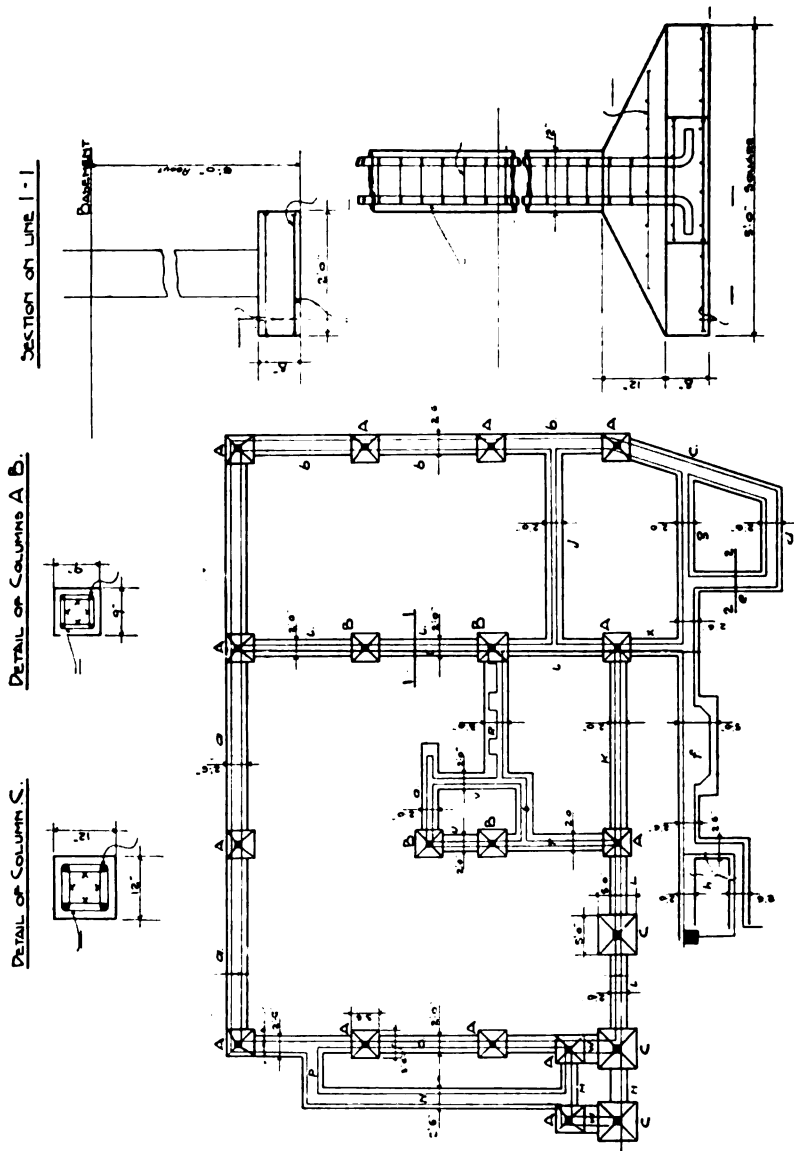


Fig. 8. Foundation and Column Details of Laundry.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.

Generally.--The floors of the wards are finished with teak boards, and the walls generally are covered with Keene's cement finished with enamel.

Many improvements have been carried out generally, apart from the construction of the new wing.

The hot-water service of the whole hospital has been centralised and con-



Fig. 9. View of Laundry.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL.

verted into the indirect system, with a view of effecting economy and obtaining higher and more uniform temperature. The heating of the new wing is by low-pressure hot water, accelerated by means of an electrically driven pump, and in the large wards this is supplemented by open fires in the centre of the room.

The radiators are of the " Hospital " type with fresh air inlets at the back fitted with internal shutters for regulating the admission of fresh air, and designed so as to be readily removable for cleaning purposes.

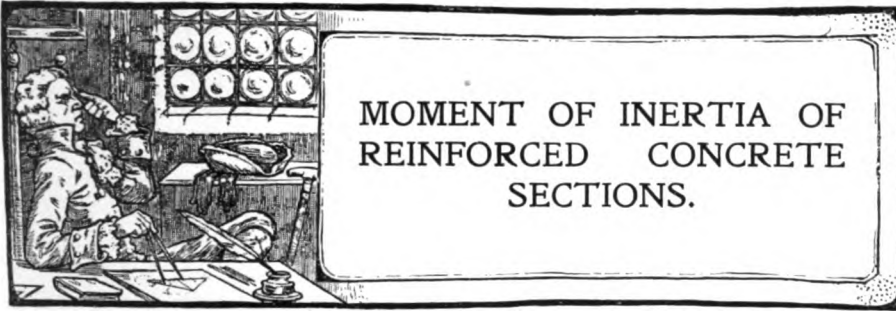
The ventilation is by natural means, consisting of opening windows and the extraction shafts previously mentioned.

A water-softening plant has been installed having a capacity of 1,000 gallons per hour. The softened water is used for the hot-water supply, laundry purposes, and boiler feed make-up. It is estimated that the introduction of this plant will save something like 50 per cent. in the cost of soap and 35 per cent. in the cost of soda.

The contractors for the reinforced concrete work and for the building generally were Messrs. Cowlin and Sons, and the heating, hot water, and water-softening installations were executed by Messrs. G. V. Haden and Sons, of Trowbridge.



Fig. 10. • View of another Corridor in West Wing.
REINFORCED CONCRETE AT THE BRISTOL GENERAL HOSPITAL



By JOHN A. DAVENPORT, M.Sc., A.M.Inst.C.E.

The following article will doubtless prove of interest to engineers and others studying various theoretical problems in connection with reinforced concrete.—ED.

THE moment of inertia is a particular case of second moment, and applies to both masses and areas. Applied to masses, it is a perfectly legitimate term, as they possess inertia; but the same cannot be said of areas, which do not possess inertia. The reason for the apparently anomalous use of the term, "Moment of inertia of an area," will be understood from the following condensed derivation of the moment of resistance of a section of homogeneous material. The complete discussion will be found in text books on Applied Mechanics and Theory of Structures, but a statement of the assumptions made will not be out of place here. They are:—

1. Plane layers remain plane.
2. Hooke's Law is true.
3. Young's Modulus the same for tension and compression.
4. The elastic limit is not exceeded.

Referring to Fig. 1, in which $A O B$ is an end view of the section considered, we have—

$$\frac{f_1}{y_1} = \frac{f}{y} \quad (1)$$

f_1 and f being stresses at points which are distant y_1 and y , respectively, from the neutral axis. Thus, at a distance y from the neutral axis the stress is—

$$f = \frac{f_1}{y_1} y \quad (2)$$

and the small area subjected to this stress can be called δA . The total force on this area is then—

$$f \delta A = \frac{f_1}{y_1} y \delta A \quad (3)$$

The moment of this force about O , a point in the neutral axis, will clearly be—

$$\delta R = \frac{f_1}{y_1} y^2 \delta A \quad (4)$$

which is the moment of resistance of the stress on the elementary part δA of the whole section. The moment of resistance of the whole section is obtained by adding up all the elementary terms similar to that shown in (4),

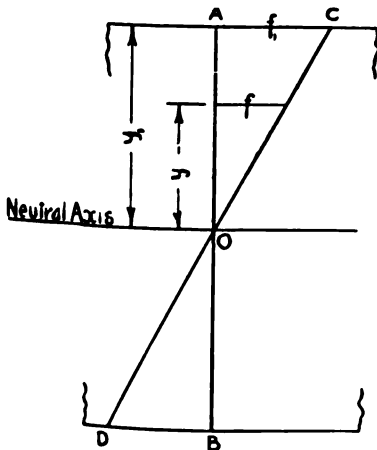


FIG. 1.

so that—

$$R = \Sigma \delta R = \Sigma \int_{y_1} f_1 y^2 \delta A = \int_{y_1} \Sigma y^2 \delta A \quad (5)$$

It should be noticed that (5) is a summation over the whole area, rendered possible by the fact that the stress line *C O D*, *Fig. 1*, is continuous. If the element δA were a mass δm , then the factor $\Sigma y^2 \delta m$ would be defined as the moment of inertia of the mass, a perfectly logical definition in view of the fact that masses possess inertia. By analogy, the factor $\Sigma y^2 \delta A$ in (5) is called the moment of inertia of the area or moment of inertia of the section, so that the moment of resistance may be written—

$$R = \int_{y_1} f_1 I \text{ or } \int_y f I \quad (6)$$

Another equation, which involves the moment of inertia, may be easily deduced, viz.—

$$\frac{R}{E I} = \frac{1}{Rad.} \quad (7)$$

in which E is Young's Modulus for the material, and *Rad.* is the radius of curvature of the beam or other piece subjected to bending.

These expressions, (6) and (7), are the fundamental flexure equations, and I , which occurs in each, is the "Moment of inertia of the section about the neutral axis." Further, the moment of inertia depends upon, and follows, the derivation of moment of resistance of the section, which, in turn, is based upon the assumptions made and already given. Shortly stated, the moment of inertia of any section must be taken about the neutral axis, and must be consistent with the assumptions made in working out the theory of bending, for both homogeneous and heterogeneous materials.

The assumptions upon which the reinforced concrete theory is based are as follows:—

1. Plane layers remain plane.
2. Hooke's Law is true.
3. The tension steel takes all the tension.
4. The bond between steel and concrete is perfect.
5. The elastic limits are not exceeded.
6. There are no initial stresses.

As a result of these assumptions it can easily be shown that, except in isolated cases, the neutral axis does not pass through the centre of gravity of the concrete section. In fact, for single reinforcement, the position of the neutral axis is a function of the amount of reinforcement; while for double reinforcement, which will be fully discussed later, it is a function of the amount of steel, its position in the compression area, and the depth of the section. Yet in dealing with the moment of inertia of reinforced concrete sections, books known to the writer all assume that the neutral axis passes through the centre of gravity of the concrete section. Taking the expression given in Appendix V. to the 2nd R.I.B.A. Report as the best known, we are given, for rectangular sections only—

$$I = \frac{1}{12} A d^3 + \frac{1}{4} (m-1) A_v d_v^2 \quad (8)$$

If we take equal tension and compression steels, as invariably the case in columns, and call r_1 either the percentage tensile or compressive steel, then—

$$\frac{A_v}{2} = \frac{r_1 b d}{100} \text{ or } A_v = \frac{r_1 b d}{50} \quad (9)$$

Substituting this value of A_v , along with $m = 15$, and $A = bd$ in (8), we get—

$$I = \frac{1}{12} b d^3 + \frac{7}{100} r_1 b d d_v^2 \quad (10)$$

and the modulus of section is—

$$S_m = \frac{1}{6} b d^2 + \frac{7}{50} r_1 b d d_v^2 \quad (11)$$

It is not the object of this article to investigate every aspect of this question of moment of inertia of reinforced concrete sections; but rather to point out the discrepancies in the expressions generally regarded at the present moment as correct; to deduce a correct expression for one particular type of reinforced section; and to compare the results obtained from the former and latter expressions. On this account a doubly-reinforced section with equal tension and compression steels will be considered, because—

Firstly, by far the larger number of column sections are of this type.

Secondly, a doubly reinforced section with equal tension and compression steels, symmetrically displaced, has the same moment of inertia both between and outside points of contraflexure, if the full area of steel be continued throughout. Thus the expression may be inserted in deflection equations. In the case of singly-reinforced beams the presence of contrary bending at any point automatically makes the moment of inertia zero, as the concrete is not assumed to be able to take any tension.

Thirdly, arches are invariably made of this section, and in the case of statically indeterminate arches it is necessary to use the moment of inertia, and

Fourthly, the doubly-reinforced section may be said to be the general case and to include also the singly-reinforced section.

Several cases present themselves depending upon the amount of cover to the reinforcement, and different conditions are provided by varying the ratio between the areas of tension and compression steels.

In the following investigation it will be assumed that the areas of tension and compression steel are each equal to A_t , and that the cover to the steel is 2 in. Using the standard notation and referring to Fig. 2 we have—

Stress in compression steel =

$$c_s = m c \frac{n-2}{n} \quad (12)$$

Considering unit width of section,

$$\text{Total push} = \frac{cn}{2} + A_t c_s = e \left[\frac{n}{2} + \frac{r_1 d}{100n} (m-1) (n-2) \right] \quad (13)$$

allowing for the reduction of the concrete area by the presence of steel.

$$\text{Total pull} = A_t t = mc \frac{(d-n)}{n} \cdot \frac{r_1 d}{100} \quad (14)$$

as $t = mc \frac{(d-n)}{n}$. Equating (13) and (14), and putting $m = 15$ and $n = n' d$, we get:

$$50 n'^2 d + 29 n' r_1 d - r_1 (15 d + 28) = 0 \quad (15)$$

A quadratic giving n' , the dimension which determines the position of the neutral axis, as a function of r_1 and d . The solution of (15) is:

$$n' = -29 r_1 d \pm \sqrt{841 r_1^2 d^2 + 3000 r_1 d^2 + 5600 r_1 d} \quad (16)$$

100d

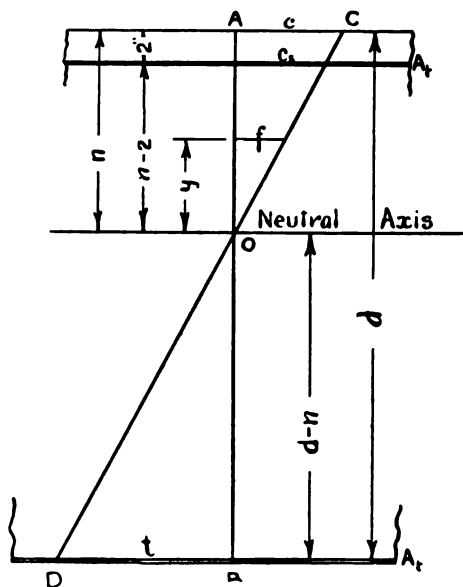


FIG. 2.

Equation (A) gives n' for all values of r_1 and d_1 with 2 in. cover to the reinforcement, but is rather awkward to use in its present form. The following expressions are much easier and sufficiently accurate for all practical purposes—

For $r_1 = 1$, i.e., 1 per cent. tension steel and 1 per cent. compression steel,

$$n' = 0.33 + \frac{0.43}{d} \quad (A_1)$$

For $r_1 = 2$,
$$n' = 0.388 + \frac{0.56}{d} \quad (A_2)$$

For $r_1 = 3$,
$$n' = 0.418 + \frac{0.62}{d} \quad (A_3)$$

For $r_1 = 4$,
$$n' = 0.435 + \frac{0.70}{d} \quad (A_4)$$

For $r_1 = 5$,
$$n' = 0.448 + \frac{0.73}{d} \quad (A_5)$$

Turning now to the derivation of moment of resistance, from which an expression for moment of inertia can be obtained, we have—

If f be the stress in the concrete at a point distant y from the neutral axis, then, see Fig. 2.

$$f = \frac{c}{n} y \quad (16)$$

and this acts on an area δA , so that the moment of the force on this area is—

$$\delta R_{c1} = y f \delta A = \frac{c}{n} y^2 \delta A \quad (17)$$

The total of such elementary moments is—

$$R_{c1} = \Sigma \delta R_{c1} = \frac{c}{n} \Sigma y^2 \delta A = \frac{c}{n} I_{cNA} \quad (18)$$

in which I_{cNA} is the moment of inertia of the concrete compression surface or area about the neutral axis. For unit width this can be shown to be $\frac{n^3}{3}$, so that

$$R_{c1} = \frac{c}{n} \frac{n^3}{3} = \frac{cn^2}{3} \quad (19)$$

As the compression steel is concentrated at a point $n-2$ from the neutral axis, the moment of this force about the neutral axis is

$$R_{cs} = \frac{(m-1) c A_t}{n} (n-2)^2 = \frac{7}{50} \frac{c r_1 d}{n} (n-2)^2 \quad (20)$$

by putting $m = 15$, $A_t = \frac{r_1 d}{100}$, and allowing for the reduction in the area of compression concrete.

The tension steel is distant $(d-n)$ from the neutral axis, and the moment of resistance is therefore

$$R_{ts} = A_t t (d-n) = \frac{3}{20} \frac{r_1 d c}{n} (d-n)^2 \quad (21)$$

The total moment of resistance of the section is

$$R = R_{c1} + R_{cs} + R_{ts}, \text{ or} \\ R = \frac{c n^2}{3} + \frac{7}{50} \frac{c r_1 d}{n} (n-2)^2 + \frac{3}{20} \frac{r_1 d c}{n} (d-n)^2 \quad (22)$$

Taking out a factor $\frac{c}{n}$, this becomes

$$R = \frac{c}{n} \left[\frac{n^3}{3} + \frac{29}{100} r_1 d n^2 - \frac{14}{25} r_1 d n + \frac{14}{50} r_1 d + \frac{3}{20} r_1 d^2 - \frac{3}{10} r_1 d^2 n \right] \quad (23)$$

This is in the same form as (6), c being the stress in the concrete at a point distant n from the neutral axis. The moment of inertia is therefore

$$I = \frac{n^3}{3} + \frac{29}{100} r_1 d n^2 + \frac{3}{20} r_1 d^3 - \frac{3}{10} r_1 d^2 n - \frac{14}{25} r_1 d n + \frac{14}{50} r_1 d \quad (24)$$

Putting $n = n'd$ in (24), we get:

$$I = d^3 \left(\frac{n'^3}{3} + \frac{29}{100} r_1 n'^2 + \frac{3}{20} r_1 - \frac{3}{10} r_1 n' \right) - \frac{14}{25} r_1 n' d^2 + \frac{14}{50} r_1 d \quad (B)$$

This is the general expression for moment of inertia, per unit width, when the covering concrete is 2 in. and the tension and compression steels are equal in area. The moment of inertia for the full breadth is given by multiplying (B) by the breadth b of the section.

Expression (B) may be simply written—

$$I = x d^3 - y d^2 + z d \quad (B_1)$$

A form which is easy to use if the factors x , y , and z be known.

The following table gives values of the factors n' , x , y , and z , calculated from the expressions (A₁) to (A₃) and (B).

TABLE OF n' , x , y , AND z .

d	n'	x	y	z
$p=1$				
10	0.373	0.09575	0.2089	0.28
20	0.352	0.09487	0.1991	0.28
30	0.344	0.09469	0.1926	0.28
40	0.341	0.09464	0.1910	0.28
50	0.339	0.09462	0.1898	0.28
$p=2$				
10	0.444	0.17712	0.4973	0.56
20	0.416	0.17477	0.4659	0.56
30	0.407	0.17435	0.4558	0.56
40	0.402	0.17418	0.4491	0.56
50	0.399	0.17404	0.4469	0.56
$p=3$				
10	0.480	0.25531	0.8064	0.84
20	0.449	0.25146	0.7543	0.84
30	0.439	0.25077	0.7375	0.84
40	0.433	0.25047	0.7274	0.84
50	0.430	0.25036	0.7224	0.84
$p=4$				
10	0.505	0.33276	1.1312	1.12
20	0.470	0.32685	1.0528	1.12
30	0.458	0.32574	1.0259	1.12
40	0.452	0.32537	1.0125	1.12
50	0.449	0.32523	1.0058	1.12
$p=5$				
10	0.521	0.40923	1.4588	1.40
20	0.485	0.40161	1.3580	1.40
30	0.472	0.40008	1.3216	1.40
40	0.466	0.39961	1.3048	1.40
50	0.463	0.39942	1.2964	1.40

As the compressive stress in the concrete at a point distant n from the neutral axis is taken in (23), it will be clear that the modulus of section will be given by dividing (B) by n or $n'd$, thus

$$S_m = d^2 \left[\frac{n'^2}{3} + \frac{29}{100} r_1 n' + \frac{3}{20} \frac{r_1}{n'} - \frac{3}{10} r_1 \right] - \frac{14}{25} r_1 d + \frac{14}{50} \frac{r_1}{n'} \quad (C)$$

the modulus of section per unit width. To get the total modulus of section multiply (C) by b , the breadth of the section.

Expression (C) may be shortly written

$$= x^1 d^2 - y^1 d + z^1 \quad (C_1)$$

the factors x^1 , y^1 , and z^1 being given in the following table for the values corresponding to x , y , and z given in the former table.

TABLE OF n' , x^1 , y^1 , AND z^1 .

d	n'	x^1	y^1	z^1
$p=1$				
10	0.373	0.25669	0.56	0.75067
20	0.352	0.26952	0.56	0.79545
30	0.344	0.27525	0.56	0.81395
40	0.341	0.27753	0.56	0.82111
50	0.339	0.27910	0.56	0.82596
$p=2$				
10	0.444	0.39891	1.120	1.26126
20	0.410	0.42012	1.120	1.34615
30	0.407	0.42836	1.120	1.37592
40	0.402	0.43330	1.120	1.39303
50	0.399	0.43633	1.120	1.40351
$p=3$				
10	0.480	0.53190	1.680	1.75000
20	0.449	0.56006	1.680	1.87082
30	0.439	0.57123	1.680	1.91344
40	0.433	0.57847	1.680	1.93995
50	0.430	0.58224	1.680	1.95351
$p=4$				
10	0.505	0.65893	2.240	2.21782
20	0.470	0.69543	2.240	2.38298
30	0.458	0.71124	2.240	2.44541
40	0.452	0.71985	2.240	2.47788
50	0.449	0.72434	2.240	2.49510
$p=5$				
10	0.521	0.78547	2.80	2.68724
20	0.485	0.82805	2.80	2.88662
30	0.472	0.84764	2.80	2.96610
40	0.466	0.85753	2.80	3.00429
50	0.463	0.86268	2.80	3.02376

The values in the foregoing tables have been used to calculate the moment of inertia and modulus of section of square sections from expressions (B_1) and (C_1) . For the purposes of comparison the same sections are treated by the R.I.B.A. expressions for moment of inertia and modulus of section. The results are given in the following table along with the error in the R.I.B.A. quantities expressed as a percentage of the correct value. It should be pointed out that as the values of d for which n' has been tabulated are 10 in., 20 in., 30 in., 40 in. and 50 in., and the cover is 2 in., the square sections will be 12×12 in., 22×22 in., 32×32 in., 42×42 in., and 52×52 in. Further, as the reinforcement will be a percentage of the area $b(b-2)$ instead of b^2 , the expressions (10) and (11) for the R.I.B.A. value of I and S_m must be modified to—

$$I = \frac{b^4}{12} + .07 r_1 b (b-2) d_v^2 \quad (25)$$

and

$$S_m = \frac{b^3}{6} + .14 r_1 (b-2) d_v^2 \quad (26)$$

It should also be noted that $d_v = b - 4$, that I is given in inches⁴, and S_m in inches³.

TABLE OF MOMENT OF INERTIA AND MODULUS OF SECTION.

p	b	I. Inches ⁴ Correct	I. Inches ⁴ R.I.B.A.	Error %	S_m Inches ³ Correct	S_m Inches ³ R.I.B.A.	Error %
1	$12'' \times 12''$	932	2,266	+143.5	250	378	+51.2
	$22'' \times 22''$	15,068	29,501	+92.5	2,143	2,682	+25.2
	$32'' \times 32''$	76,514	140,065	+83.1	7,416	8,754	+18.1
	$42'' \times 42''$	242,028	429,122	+77.3	17,744	20,434	+15.2
	$52'' \times 52''$	591,084	1,028,629	+74.0	34,870	39,563	+13.5
2	$12'' \times 12''$	1,596	2,803	+75.7	359	467	+30.0
	$22'' \times 22''$	26,906	39,480	+46.8	3,234	3,589	+11.0
	$32'' \times 32''$	138,035	192,751	+39.7	11,306	12,047	+6.6
	$42'' \times 42''$	438,640	598,937	+36.6	27,232	28,521	+4.7
	$52'' \times 52''$	1,073,842	1,447,957	+34.8	53,841	55,691	+3.4
3	$12'' \times 12''$	2,197	3,341	+52.1	458	557	+21.7
	$22'' \times 22''$	37,989	49,459	+30.2	4,230	4,496	+6.3
	$32'' \times 32''$	196,206	245,436	+25.1	14,900	15,340	+3.0
	$42'' \times 42''$	625,624	768,751	+22.9	36,122	36,607	+1.3
	$52'' \times 52''$	1,535,898	1,867,285	+21.6	71,393	71,819	+0.6
4	$12'' \times 12''$	2,770	3,878	+40.0	549	646	+17.8
	$22'' \times 22''$	48,754	59,438	+21.9	5,187	5,403	+4.2
	$32'' \times 32''$	252,969	298,121	+17.9	18,412	18,633	+1.2
	$42'' \times 42''$	807,181	938,566	+16.3	44,725	44,694	-0.07
	$52'' \times 52''$	1,985,984	2,286,613	+15.1	88,468	87,947	-0.6
5	$12'' \times 12''$	3,328	4,416	+32.7	639	736	+15.2
	$22'' \times 22''$	59,349	69,417	+17.0	6,118	6,311	+3.1
	$32'' \times 32''$	308,951	350,805	+13.6	21,819	21,925	+0.5
	$42'' \times 42''$	988,638	1,108,380	+12.1	53,048	52,780	-0.5
	$52'' \times 52''$	2,431,416	2,705,941	+11.3	104,989	104,075	-0.9

It may be urged that as the expressions (A_1) , (A_2) , (A_3) , (A_4) and (A_5) are not exact, the values of I and S_m , given in the table as correct, are not correct. But the object of

this article is not so much to give accurate values of these quantities, as to point out the errors in the R.I.B.A. expressions and the reasons for such errors. Nevertheless, the writer has taken all possible care, and hopes that the values given are as accurate as slide-rule calculations and ordinary care will permit; in fact, they may be said to be "practically" correct for the range taken.

An inspection of the table shows several interesting things as follows:—

Firstly, that the error in the modulus of section is less than that in the moment of inertia, for all values in the range taken, because the R.I.B.A. divisor, $\left(\frac{b}{2}\right)$, is greater than the correct divisor, $[n'(b-2)]$.

Secondly, that for a fixed percentage reinforcement both errors vary, getting less as the section increases.

Thirdly, that for any section the errors get less as the percentage reinforcement increases.

Fourthly, that for any fixed percentage reinforcement the errors decrease with the size as above and appear, although the results do not reach it, to approach a lower limit which is not zero.

Fifthly, in the higher percentages and larger sizes the error in the modulus of section changes from positive to negative.

If results can be obtained without using the moment of inertia the errors are not of such great importance; but in the case of a statically indeterminate arch, for example, it is necessary to use I , and the errors, or a consideration of them, therefore become important. For such an example we have by the elastic theory the summation

$$\sum \frac{Bx \delta s}{EI}, \quad \sum \frac{By \delta s}{EI}, \quad \text{and} \quad \sum \frac{B \delta s}{EI},$$

while if the principle of least work be used we have the summation

$$\sum \frac{B^2 \delta s}{2EI}$$

In working out these cases, by a semi-graphical method, some writers recommend the taking of sections for which $\frac{\delta s}{I}$ is constant. If 1 per cent. reinforcement be used with the R.I.B.A. form and the depth of the section is 22 in. or less, then an error of about 100 per cent., or more, in I , will be communicated to $\frac{\delta s}{I}$, and consequently the chosen sections will be incorrect, possibly to a very large extent. Further, as the error varies with the depth, the error in the sections will vary with the depth, and cannot be rectified by a simple factor.

For calculating deflection of beams and cantilevers, equation (7) leads to an expression as follows—

$$\delta = C \frac{W l^3}{EI}$$

in which C is a constant which depends upon the type of beam and loading. If such a form of equation be used for calculating deflection or any other quantity for reinforced concrete beams, it will be seen from the foregoing tables of I that the calculated result may easily be over 100 per cent. in error if the R.I.B.A. expression for I be used and the section be similar to those already considered. Results deduced from such calculated values will be largely in error and may lead to serious diminution in the actual factor of safety.

When the usual assumption, that the compression steel lies at the centre of pressure of the concrete compression is used, the expressions for I and S_m are much simpler than those given.

RECENT BRITISH PATENTS RELATING TO CONCRETE.

We propose to present at intervals particulars of British Patents issued in connection with concrete and reinforced concrete. The last article appeared in our issue of January, 1915.—ED.

Reinforced Concrete Piles.—No. 16301/14. C. I. Deane, Walla Walla, Washington, U.S.A. Accepted December 24/14.—Reinforced concrete piles constructed in



Fig. 5. REINFORCED CONCRETE PILES.

accordance with this invention comprise a central metallic core (10) in the form of a tapered tube of suitable length, and a shell (11) of concrete, provided with an optional additional reinforcement (12). The head (13) consists of a steel ring riveted or otherwise rigidly fixed to the upper end of the core and having a flange (15) which gives a bottom recess (14).

The bottom of the pile has a steel driving shoe (16) which has a top recess (17) at the centre of which is an upstanding threaded stud (18) which is screwed into the lower end of the core (10).

Eye-bolts (18¹) are screwed into the core (10) and project from the surface of the pile a sufficient distance to permit the attachment of stays and lateral bracing. The shoe may have a vertical aperture if the jet system of driving is employed.

If a tubular core is employed, it may be filled with concrete after the pile has been driven, leaving enough of the core unfilled to permit the insertion of a post for the connection to the superstructure.

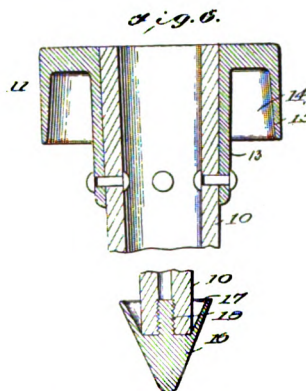


Fig. 6. REINFORCED CONCRETE PILES.

Concrete Walls.—No. 11110/14. G. S. Mumford, 1328 Broadway, New York, U.S.A. Accepted November 26/14.—Concrete walls are constructed in accordance with this invention by first forming piers at intervals at the base of the wall, erecting movable guideways on these piers, and constructing permanent columns between some of the guideways; the wall is then cast progressively between the columns by moulding devices sliding on the guideways.

The piers (2), Fig. 1, are formed with vertical rabbets or dovetail grooves, indicated by dotted lines, and rise to a height of 6 or 8 ft., being placed about 20 ft. apart. A wooden framework or guideway (3), comprising braced beams (4, 5) bolted together by bolts (6) passing through pipe-spacers (7), is erected on each pier, the moulding devices (12, 13) being carried by pulley tackle from a transverse member (10), lateral bracing being provided if required.

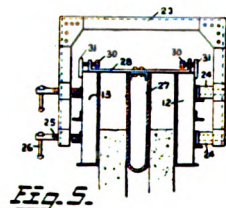
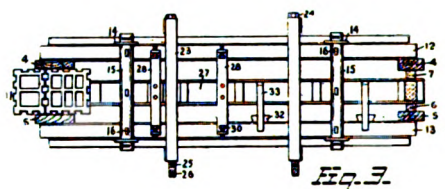
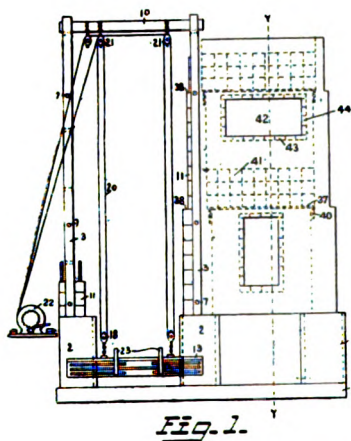
After the framework (3) has been erected, hollow-tile columns (11), which may be filled with cement and reinforcing rods and have metal lugs for bonding the wall fixed in their horizontal joints, are built between the beams (4, 5). The bracing of the latter is removed as the columns progress, the pipe-spacers (7) being allowed to remain.

The moulding device comprises two flat members (12, 13) about 20 ft. long and 2 to 4 ft. high, which are formed preferably of wood lined with sheet metal, and are

braced with angle bars. Clamps (23), *Fig. 5*, are provided at two or more points along the mould to prevent bulging of the sides.

For hollow walls a rectangular, slightly-tapered sheet-metal core (27) is supported by cross-bars (28) from the moulds (12, 13), and is centred by means of screws (30) engaging lugs (31). Additional cores may be employed for making grooves to facilitate the fixing of a brick facing when employed.

To facilitate the insertion of reinforcement over openings, the columns are formed



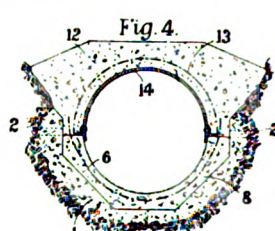
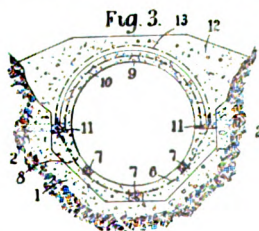
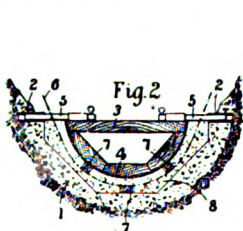
Figs 1, 3, 5. CONCRETE WALLS.

with offset portions (38) into which the down-turned ends (40) of the reinforcement are inserted.

When inserting doors and windows (42), *Fig. 1*, the core (27) is removed and a brick or tile foundation is laid on the concrete; the window frame is then placed on this foundation and surrounded with brick (44).

In any given position of the moulds (12, 13) the concrete is poured in, and after setting the mould boards are moved upwards, allowing a lap of 5 or 6 inches, and then clamped again in position and the operation repeated.

Concrete Conduits, Sewers, etc.—No. 1124/14. *E. R. Calthrop, Eldon Street House, Eldon Street, London, and F. G. Lynde. Accepted January 14/15.* In constructing a conduit, sewer, etc., in accordance with this invention, a trench is first excavated and lined with concrete (1), *Fig. 2*. Suitable mould-supporting means, such as pipes (2), are inserted into the concrete at appropriate distances apart, and moulds (3), appropriately built up of timber and lined with metal (4), are suspended by bolts (2).



Figs. 2, 3, 4. CONCRETE CONDUITS, SEWERS, ETC.

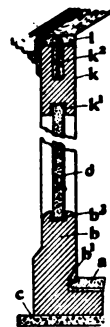
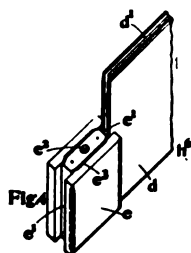
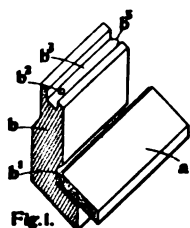
Expanded metal or other suitable reinforcement (6) is then placed in position and held in place by the bolts (5), distance pieces (7) of concrete or other suitable material being advantageously employed. The space between the mould and the lined trench is then filled with concrete (8), and the bolts are loosened and the moulds removed when

the concrete has set, thus leaving one half of the conduit completed. A reinforced concrete cover (9), *Fig. 3*, previously formed, is then cemented in position and the whole is filled in with concrete (12), additional reinforcement (13) being employed if desired.

In a modified construction, *Fig. 4*, a metal plate (14) of the requisite shape is placed over the half conduit, these plates forming a permanent part of the construction.

In a further modification the reinforcement (6) is replaced by a specially wound rod covered with expanded metal; when the lower portion has set, the concrete cover (9) or plate (14) is slipped from the end into position under the spiral reinforcement.

Concrete and Stone Buildings.—No. 14805/13. C. H. Provis, Mutley Road, Mannamead, Plymouth. Accepted June 18/14.—This invention comprises an improved construction of base blocks and vertical pillars for buildings formed of recessed blocks and pillars adapted to receive the edges of wall and floor slabs. The base blocks (b) are formed with recesses (b¹) to receive the floor slabs (a) and are provided with vertical



FIGS. 1, 4, 9. CONCRETE AND STONE BUILDINGS.

holes (b²) for the insertion of iron rods which are preferably grouted in and serve to support the pillar blocks.

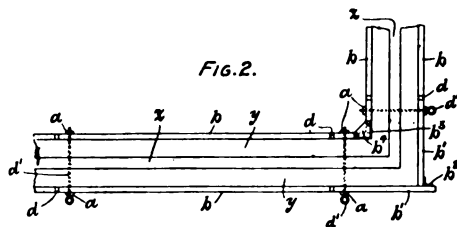
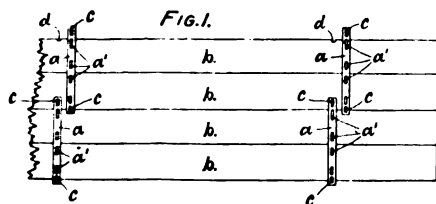
The pillar blocks (e) are formed with grooves (e¹) in their vertical edges for the reception of the wall slabs (d), which may be provided with grooves (d¹) to ensure a good joint. Vertical holes (e²) are formed through the pillar blocks in register with the similar holes in the base blocks.

Vertical recesses (e³) in the pillar blocks are for the grouting of successive blocks.

Special blocks having the features indicated above are described for corners and door frames.

Hollow Concrete Walls.—No. 28799/13. W. J. Ayles, Broughton, Stockbridge, Hants. Accepted December 10/14.—According to this invention hollow concrete walls are made from slabs or by casting concrete in two independent parts, each part being provided with ribs disposed in two or more directions.

When the walls are cast *in situ*, shuttering, comprising boards (b), *Fig. 1*, connected by metal plates (a), is employed. The plates have apertures (c) and bolt-slots (a¹),



FIGS. 1 & 2. HOLLOW CONCRETE WALLS.

and project beyond the edges of the boards, which are provided with notches (d) for the passage of bolts (d¹).

Cores, supported as described in Specification No. 29844/12, are employed for shaping the interior of each part of the wall.

At the corners, *Fig. 2*, angle plates (*b2*) connected to portions (*b1*) hold the boards (*b1*) together, inside moulding boards (*b3*) connected by plates (*b4*) being employed.

For making slabs or blocks, the forms shown in *Fig. 10* are employed. The boards (*b*) are fixed at greater distance apart and partitions (*f*) are employed to divide the blocks, spacing members (*h*) being employed; the partitions (*f*) have projections (*j*) for positioning the spacing members. Cores (*t*) are attached to one or both sides of the partitions (*f*). The mould is held firm by means of angle plates (*f₂*, *f₃*).

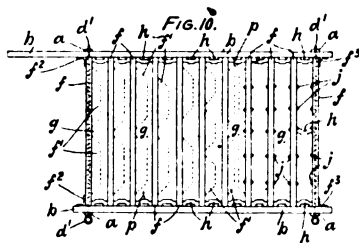


Fig. 10. HOLLOW CONCRETE WALLS.

Reinforced Concrete Buildings.—No. 7210/14. G. B. Waite, 31st Street and East River City, New York, U.S.A. Accepted October 29/14.—According to this invention the steel of a reinforced concrete building forms a self-supporting skeleton frame which is erected ahead of the concrete and gives the lines for the erection of the forms which are hung to and temporarily supported by the frame.

The steel floor members are small I-beams of standard commercial shapes, capable of holding themselves in straight lines and stiff enough to brace the temporary structure and support necessary scaffolding to permit the steel construction to proceed ahead of

members are so located that they will act ultimately as tension members for the concrete, shear-
ing strips or bars being also preferably included; but until the concrete is hardened the steel acts independently and carries the temporary load.

The columns (11), *Fig. 1*, may be a skeleton form, and are formed of light steel bars of commercial shapes disposed vertically, and preferably spaced and connected by lattice work and a filling and covering of concrete, or they may be formed of steel members connected together, as shown at (14), and enclosed in concrete, or of other suitable form.

Light steel I-beam girders (18, 19) of single or twin formation respectively extending between and supported by the columns and steel cross-beams (20) are disposed transversely of and supported on the girders (18). The light steel girders and floor beams constitute in the finished structure the steel reinforcement of the finished concrete beams.

The steel floor members need be frame structure together and hold
ding loads.

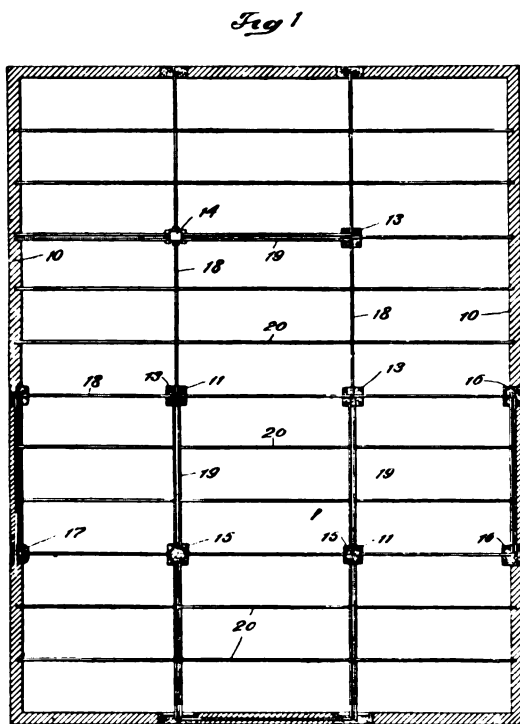


Fig. 1. REINFORCED CONCRETE BUILDINGS.

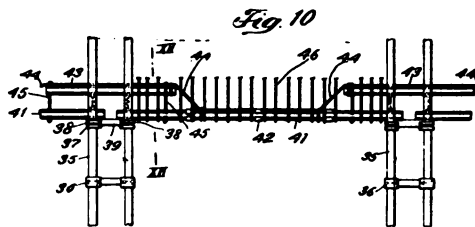
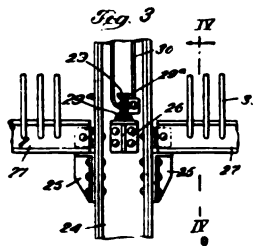
of sufficient weight and strength only to tie the light frame structure together and hold up scaffolding and other normal and temporary building loads.

When the built-up steel columns, such as shown at 14 in *Fig. 1*, are used, the girders are connected to brackets supported by the columns substantially as shown in *Fig. 3*, and the columns and girders and floor beams are then encased in concrete and serve as reinforcements therefor.

Each girder and floor beam, various forms of which are described, carries upward extending U-shaped shear bars (30) which extend near to the surface of the concrete floor.

In the construction shown in *Fig. 10*, four angle bars (35) are employed, each forming one corner of the rectangular column structure. These bars are connected together at their ends by means of clamps (36), and somewhat below the floor level and approximately on the ceiling level anchoring clamps (37) are secured to the angle irons of the columns forming temporary supports for the floor structure.

Bars (39) carried by the clamps form supports for the floor beams (41), which



Figs. 3 & 10. REINFORCED CONCRETE BUILDINGS.

extend within the column any suitable distance, and for that reason need not be accurately cut to proper lengths. These floor beams are preferably made up of two channel iron members separated a suitable distance from each other for the greater portion of their length, these separated members being curved inwardly or converging towards their ends, so that their ends within the column frames are closer together, these bars being held by separators at the requisite distance apart.

Short beams (43) are provided to take up the negative bending movement over the points of support for the floor beams, and bars (44) connect the upper and main reinforcements.

To take up the shear stresses, shear bars (46) are bent around the floor beams (41) and are carried to a point slightly above the reinforcing beams (43); these shear bars adjacent the columns embrace the reinforcing beams.

Concrete Reinforcements.—No. 39/14. *F. A. Macdonald and Partners*, 135, *Wellington Street, Glasgow*. Accepted November 5/14. —This invention relates to steel rods or bars for use in reinforced concrete work of the type in which a series of uniformly arranged circumferential or like projections are provided to ensure accurate spacing of the auxiliary reinforcements along the length of the bar and to provide a mechanical bond of the main bar with the concrete. According to the present invention the projections take the form of twin projections or ridges having the intervening space or groove adapted to exactly fit the auxiliary reinforcement, so that slipping in either direction is prevented. The invention also provides improved rigid seat plates or spacers for spacing apart reinforcing rods, and for affording connections for shafting brackets and similar attachments to concrete beams and other members.

The continuous bar (a) has raised twin circumferential projections or ridges (b, b'), which are spaced at regular intervals along the length of this bar, these projections being formed during the rolling of the bar.

The twin circumferential projections form a series of abutments, usually 3 in. apart, along the full length of the bar with a locking space (c) between them, in which space auxiliary reinforcements required for diagonal tension in reinforced concrete beams and for compression stresses in reinforced concrete columns and piles are placed, and the twin projections (b, b') securely lock the auxiliary reinforcements against longitudinal slip along the main central bar.

Fig. 3 shows a modification in the form of helical twin projections or ridges (b²).

Fig. 6 shows the use of the bars with diagonal shear members of variable spacing, but helical bars may be employed.

The lock spacer (*s*), *Fig. 8*, is adapted to take six rods or bars (*a*) in two horizontal tiers of three bars in each tier in the lower part of the part metallic truss. These lock spacers are made with spaces in the form of three internal triangles, the centre triangle

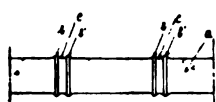


FIG 2

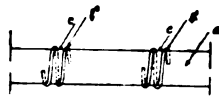


FIG 3

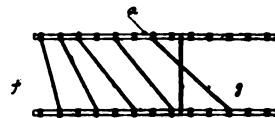


FIG 6



FIG 8

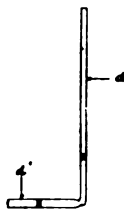
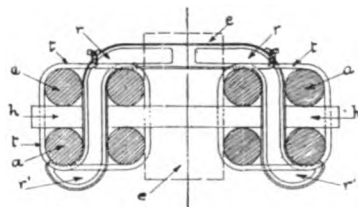


FIG 9



Figs. 2, 3, 6, 8, 9 and 14. CONCRETE REINFORCEMENTS.

being formed in inverse position to that on either side of it, and each angle of the triangle is rounded to such radius as is required to receive the bar (*a*) of the truss.

On the rods or bars (*a*) and lock spacer (*s*) being adjusted to their correct positions, as shown at *Fig. 12*, a lever (*d*) of the form shown at *Fig. 9* is inserted between the upper and lower tiers of bars (*a*), and by turning the lever and inserting two or more key pieces (*h*) the various bars (*a*) forming the part metallic truss are locked into position.

A modified form of spacer is shown in *Fig. 14*. The two inner bars of the lower tier are first placed in the positions shown, and a horizontal key (*h*) (a short length of round iron of the exact requisite diameter and length) is then superimposed on these bars, and is then placed in position, and a temporary vertical key (*e*) (of wood or other suitable material) may then be arranged to hold the vertical tiers of bars apart from each other.

The two outer bars of the lower tier are then placed within the curved ends of the spacer (*r*), and the two outer bars of the upper tier are placed in position. The binders (*t*) (made from stout annealed wire or other suitable material) are then placed in position along the length of the lower part of the truss as required, and the temporary key (*e*) is thereafter withdrawn.

This spacer (*r*) enables the whole of the eight bars to be supported by means of the "bridge" provided by the horizontal key (*h*), the ends of which are supported on certain of the bars (*a*), which bars are again supported on the curved ends (*r*) of the spacer, which owing to its form also serves to hold the bars apart from each other, and on the binders (*t*) being placed in position a truss containing eight bars is formed by means of the spacer, in which the bars are rigidly held apart from each other in the lower area of the beam, and which can be handled and transported without difficulty or distortion of the component members.

For attaching shafting brackets and other connections to reinforced concrete beams

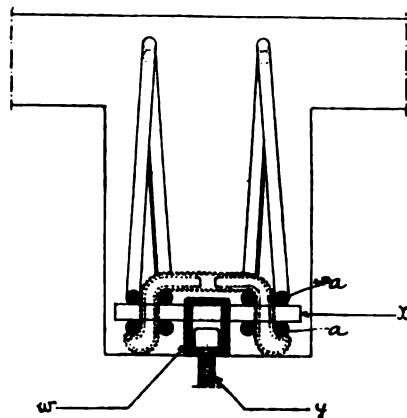
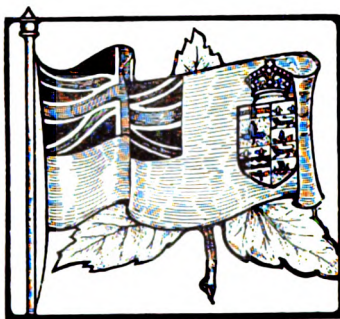


FIG 28

Fig. 28. CONCRETE REINFORCEMENTS.

in combination with trusses formed by means of the spacers, a channel (*w*) (*Fig. 28*) formed from sheet metal is provided, with holes in the sides opposite to each other at such intervals as are required, and the channel is then secured to the trusses by keys (*x*) passed through these holes and between the upper and lower tiers of bars (*a*) of the trusses, and by this means loads may be suspended from the beam in any position required, as the channel is efficiently secured and attached by the steel keys (*x*) to the steel reinforcement of the reinforced concrete beam. Movable screwed studs (*y*) may be placed within the channel (*w*) to any number and in any position required, forming the means of movable attachment for shafting brackets and other connections.



**HALIFAX OCEAN
TERMINALS.
NEW PIER No. 2.
INTERCOLONIAL RAILWAY OF
CANADA DEEP-WATER TERMINALS,
HALIFAX, N.S.**

By A. F. DYER, A.M.Inst.C.E.

The following particulars regarding the construction of the New Pier at Halifax are specially interesting on account of the very large piles driven.—ED.

TOWARDS the end of 1910 the Department of Railways and Canals of Canada decided to enlarge and improve the shipping terminals of the Intercolonial Railway at Halifax, N.S., in order that the rapidly increasing transatlantic shipping making use of that port might be conveniently accommodated.

The shipping piers then owned by the railway at their "deep-water" terminals consisted of a flour-loading pier, 470 ft. long by 50 ft. wide, which had been built by Sir Samuel Cunard before he left Halifax to found the Cunard Shipping Company; old Pier No. 2, 560 ft. long by 65 ft. wide, having a two-storey wooden shed, and used by incoming passenger and mail steamers; Pier No. 3, 610 ft. long by 165 ft. wide, with a one-storey freight shed, used by incoming freight and outgoing passenger and freight steamers; and Piers 4 and 5, used for the shipping of coal and lumber. All these piers being supported on timber piles, and having wooden floors, require a considerable outlay for maintenance, for although the Teredo is practically unknown in Halifax Harbour, the Limnoria is very active.

In the spring of 1911 the following scheme of development was put forward by Mr. J. Kennedy, Consulting Engineer to the Montreal Harbour Commissioners, and approved of by the Department of Railways and Canals: A new pier, 800 ft. long by 235 ft. wide, to be built on the old Cunard property in place of the Cunard Pier, old Pier No. 2, and the Immigration Building, all three of which were to be demolished, the last two on the completion of the new pier, which would then be known as New Pier No. 2; Pier No. 3 was to be lengthened to 800 ft. and widened southwards to a width of 235 ft., leaving a dock 380 ft. wide between its south side and the north side of New Pier 2; a new pier, No. 4, 800 ft. long by 235 ft. wide, to take the place of existing Piers 4 and 5, with a basin 275 ft. wide on the north of Pier 3; and a fourth pier, 700 ft. by 195 ft., 250 ft. north of Pier No. 4. This scheme provided for about 6,000 lineal ft. of berths for ocean-going steamers, against the 3,320 ft. then existing.

It was decided to start work with the building of New Pier No. 2, as the construction of this pier would least interfere with the existing accommodation.

In designing the new piers, the two considerations of economy of con-



Fig. 1. Site of Pier looking South.
OCEAN TERMINALS, HALIFAX, NOVA SCOTIA

struction and permanency of the structure determined the type to be used, which is that of a reinforced concrete floor supported on reinforced concrete piles. This type of construction was well suited to the local conditions of the site, where the depth of rock-bottom varied from 42 ft. to 68 ft. below low-water level of spring tides, which necessitated a substructure below the pier floor of from 61 ft. to 87 ft. in height.

Borings taken over the site of the new pier indicated that a layer of hard clay or hard pan covered the rock to a depth which varied from 2 ft. to 12 ft. Above the hard pan there was found a layer of soft mud, which had a depth of nearly 30 ft. at the shore end, but ran out to about 5 ft. at the outer end of the proposed pier.

NEW PIER No. 2.

The elevation of the pier deck was determined by the existing tracks in the railway yard, and on the other piers at 19 ft. 2 ins. above extreme low-water level of spring tides, which is 3 ft. 6 ins. above top of rail. Owing to the small height of the structure above low water compared with the depth of the foundations below low water, a system of diagonal transverse bracing would not have greatly increased the stability of the pier and its ability to withstand the shock of a large vessel striking it on one of its sides, and therefore recourse was had to the use of reinforced concrete brace piles driven at an angle of one-in-three to the vertical and having their heads consolidated with the transverse girders of the pier floor. It is believed that this is the first instance of such a system of bracing being used for a reinforced concrete pile pier. Further stiffening of the structure was obtained by forming a bank of dredged material under the outer end of the pier where the depth of water was greatest and the thickness of the material overlying the rock was least.

The question of the durability of reinforced concrete piles immersed in sea water, in a climate such as prevails during the winter months in Halifax, had now to be dealt with. The large ratio between the surface exposed and the bulk of the concrete in this type of construction is one of its weak points, and special measures had to be taken to prevent any deteriorating action of the water on the concrete.

The measures taken to prevent as far as possible any injurious actions taking place on the concrete piles were:—

The Portland cement used for the making of all concrete below high-water level did not contain more than 6·3 per cent. of alumina.

All concrete below high-water level was mixed in the proportions of 1 part cement, $1\frac{1}{2}$ parts sand, and 3 parts crushed quartzite gravel, with the object of obtaining a rich and as dense and non-porous a concrete as was possible.

All concrete surfaces from low-water level to about 2 ft. above high water were sheathed with two layers of 2 ins. creosoted pine planking bedded on cement mortar. The inner surfaces of the planks were coated with pitch to prevent any deteriorating action taking place between the creosote in the wood and the concrete of the piles. This sheathing keeps the concrete surfaces moist, prevents all frost action, and protects the concrete from abrasion by ice as well as the wearing action of the wash of the water.

The plans included the erection of a two-storey freight shed 200 ft. wide

and extending from the bulkhead line to within 15 ft. of the outer end of the pier. This shed allows room to run a railway track along each edge of the pier. Two additional railway tracks run the whole length of the shed down the centre of the pier, dividing the lower floor into two separate freight floors each 88 ft. in width. The columns supporting the shed are spaced 39 ft. 6 ins. transversely and 18 ft. longitudinally, the long transverse span being decided upon in order to facilitate the handling of gangways.

The loads for which the pier and shed floors are designed are: Pier deck, 1,000 lb. per sq. ft. live load; track girders, 4,500 lb. per running foot of each rail live load; upper floor of shed, 500 lb. per sq. ft. live load; and roof, 110 lb. per sq. ft. total load.

The longitudinal spacing of the shed columns determined the distance



Fig. 2. View of Shed Forms.
OCEAN TERMINALS, HALIFAX, NOVA SCOTIA.

between each bent of piles, and it was decided to use piles capable of supporting a load of at least 100 tons, including their own weight in water. This necessitated the use of piles 24 ins. square in section, and required thirty-three vertical piles and six sloping piles in each bent. Two vertical piles and one sloping pile are driven beneath each side column of the shed, three vertical piles and one sloping pile beneath each interior column, and the remainder of the vertical piles are spaced out between the column groups at approximately 10 ft. centres. Of the six bracing piles three are driven with heads inclined to the north side of the pier and three to the south.

The amount of reinforcement in the piles varied in accordance with the length of the pile as cast, and was fixed by the amount required for their safe

handling when being placed in position for driving. No metal shoes were used. Some additional stirrups were placed near the head of the piles to prevent excessive breakage while driving, and the longitudinal rods were allowed to project 3 ft. beyond the concrete to save the labour of cutting away the concrete in order to secure the necessary bond of the pile reinforcement with the concrete of the pier deck. In order to minimise the bending moments in the piles during handling, it was specified that all piles were to be lifted in such a way that they were suspended at points one-fifth of their length from each end while in a horizontal position. By the observance of the above rule the cracking of the piles while being conveyed from the moulding yard to their ultimate position ready for driving was entirely eliminated.



Fig. 3. Pile Driver.
OCEAN TERMINALS, HALIFAX, NOVA SCOTIA.

The piles to be driven as brace piles were reinforced in a similar manner to the vertical piles, but were cast with a camber, so that when built into the pier and under an axial load of 80 tons, the stresses on the cross-section of the concrete, due to this load, and the bending moment of its own weight would be uniform. The number of concrete piles used in the work is 1,801, 238 of which were driven at an angle as bracing piles.

The pier deck is built entirely of reinforced concrete, carried on reinforced concrete girders 36 ins. deep, which run transversely across the pier along the pile bents, and on girders of the same depth running longitudinally at an average of 9 ft. 10½ ins. centres. The 18 ft. by 9 ft. 10½ ins. rectangle thus

formed is divided again by a transverse floor beam 24 ins. deep. The slab is 8 ins. thick, reinforced in both directions with $\frac{1}{2}$ -in. round rods at 7-in. centres, every alternate rod being bent up over the supporting beam.

All beams and girders are designed as T-section beams, sufficient web

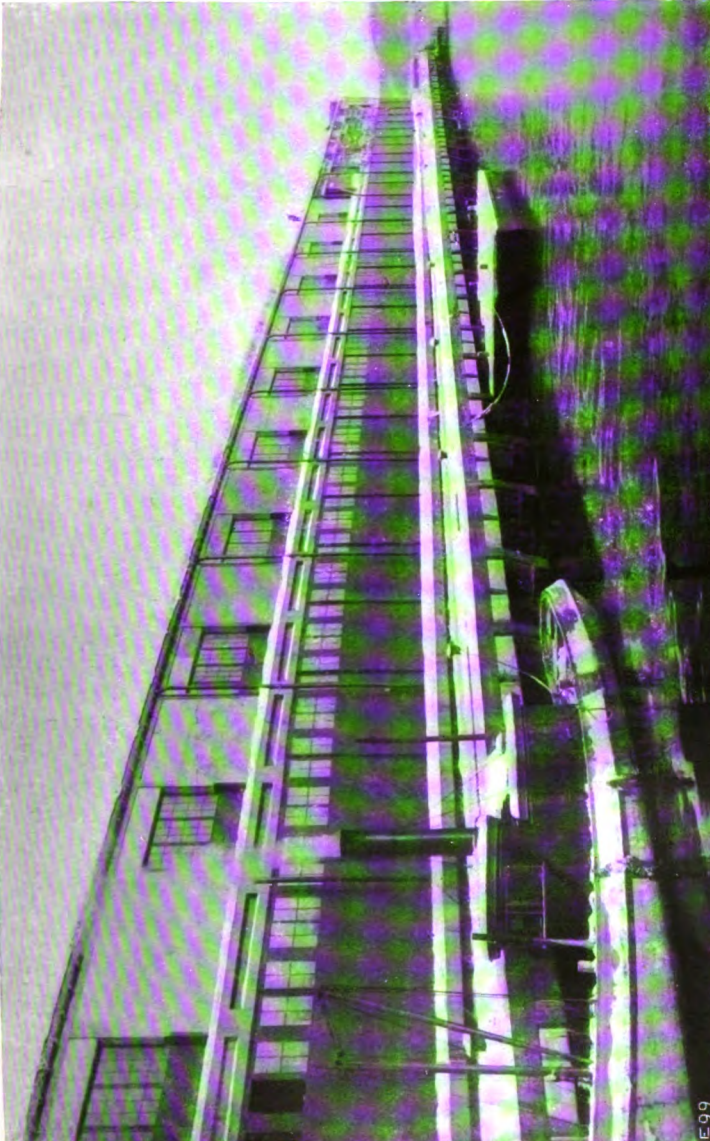


Fig. 4. South Side of Pier.
OCEAN TERMINALS, HALIFAX, NOVA SCOTIA.

reinforcement being used to take care of the excessive shearing stresses, the top ends of the U-stirrups being bent horizontally and carried into the slab as far as the table of the T extends, in order that the shearing stresses occurring between the web and table be properly cared for.

Owing to the side and central railway tracks being carried at a level 3 ft. 6 ins. below that of the pier deck proper, transverse concrete gussets were built in at the four breaks in the floor level, in order that the rigidity of the floor system would be maintained from one side of the pier to the other, and the whole floor would act as a monolith in resisting the shocks of large vessels.

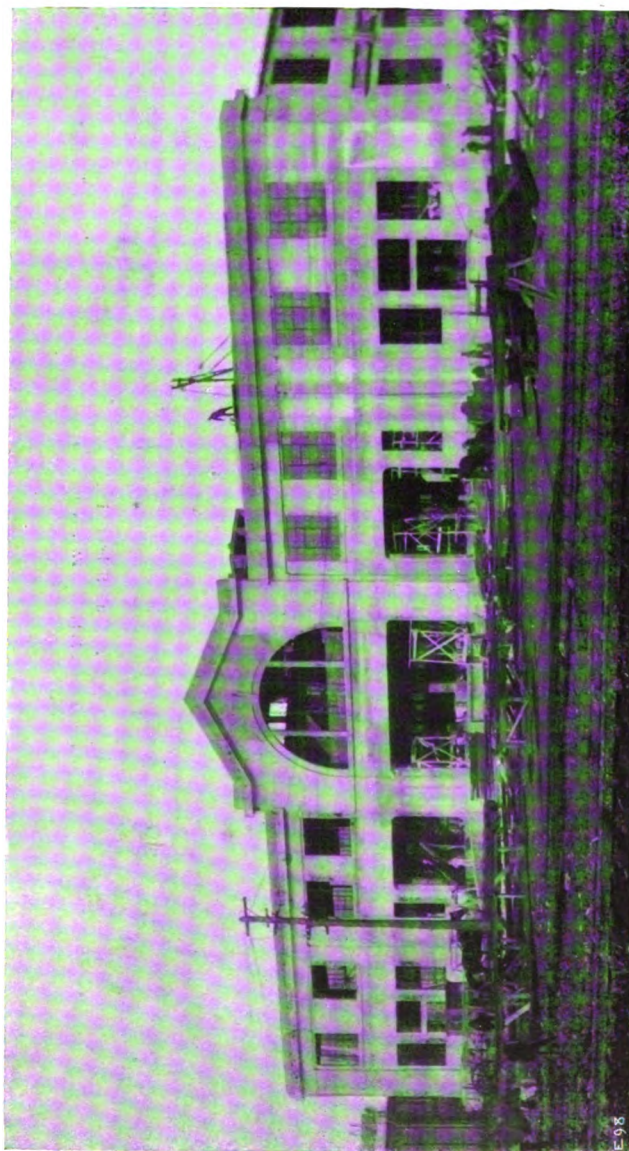


Fig. 5. West End Wall of Shed.
OCEAN TERMINALS, HALIFAX, NOVA SCOTIA.

A concrete beam 51 ins. deep is carried along each edge of the pier, and to the outside of this is secured a rubbing fender of pitch pine with a vertical face of 20 ins. At the outer corners of the pier the depth of the fender beam was increased to about 11 ft., the corners being cut diagonally across, leaving a flat concrete surface 16 ft. wide by 11 ft. deep, strongly braced at the back by gussets from the pier floor. Numerous holes were left in this concrete face for attaching thereto fenders made up of birch or willow saplings bound

together into bundles. Two layers of bundles are attached to each corner, the larger ones towards the middle so as to form a rounded fender, the whole being bound in securely with three sets of steel cables.

Cast iron mooring posts of the mushroom-head type are placed at 54 ft.

centres along each side and outer end of the pier. Each post is anchored down with eight $1\frac{1}{2}$ -in. round bolts, 30 ins. long, built into the concrete deck.

The interior columns supporting the upper floor of the shed are circular in cross-section, $25\frac{1}{2}$ ins. in diameter, reinforced with $1\frac{1}{8}$ -in. vertical rods and $\frac{1}{4}$ -in. hooping at 1-in. pitch. The side columns are rectangular in section, 16 ins. by 20 ins., reinforced with vertical rods with $\frac{1}{4}$ -in. lacing at 6-in. centres.

The upper floor beam system consists of transverse girders supported on the six columns of each bent, and longitudinal beams at 9 ft. $10\frac{1}{2}$ in. centres carrying a concrete slab 7 ins. in thickness. The transverse girders have a span of 39 ft. 6 ins. centre to centre of columns. In order to reduce the height of the upper floor above the pier deck to a minimum, and at the same time to have adequate headroom for the lower storey, the depth of the girders was made 30 ins. and the thickness of the floor slab was increased to 9 ins. for a strip of 18 ins. along each side of the girder to provide sufficient compression area.

The roof consists of tar and gravel roofing laid on boarding and carried on a light reinforced concrete slab. The roof beam system consists of 12 in. by 42 in. transverse girders carried on

12 in. by 12 in. concrete posts, and 8 in. by 12 in. longitudinal beams, with spans the same as the upper floor. The centre bay of the roof is raised 5 ft. for the total length of the building, thus forming a clerestory and providing good light for the upper storey.

Steel sliding doors enclose each side of the lower storey, except for a length of 72 ft. at the shore end, which is given over to shipping and railway offices. On the upper floor a 17-ft. wide opening, closed with two steel

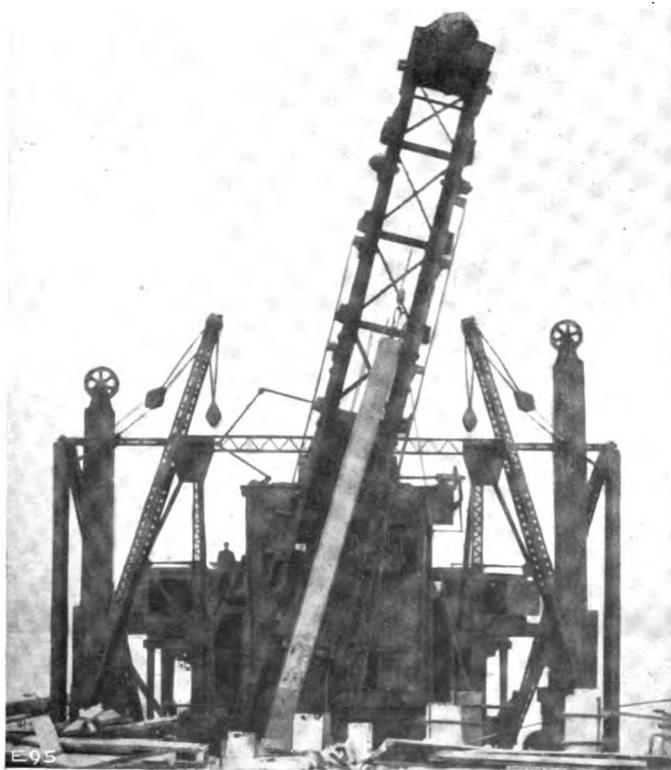


Fig. 6. Brace Pile ready for lowering into position.



FIG. 7. View of Pier looking East.
OCEAN TERMINALS, HALIFAX, NOVA SCOTIA.

sliding doors, is provided at 36 ft. centres. All sliding doors for both floors are provided with windows glazed with ribbed-wire glass.

In order to secure additional yard room at the base of the pier, the old bulkhead of the Cunard pier had to be abandoned and a new bulkhead built. This new bulkhead was built 250 ft. east of the east side of Water Street, and was formed of timber cribwork filled with rock and sand. After the commencement of the work it was decided to further widen the yard, and an additional 106 ft. of yard room was obtained by the construction of a reinforced concrete pile wharf similar in construction to the pier deck. The length of the pier was thus reduced to 694 ft. from the new bulkhead line.

Construction Work.— The contract for the construction of the pier and shed was let to the Nova Scotia Construction Company, Ltd., of Sydney, C.D., in September, 1911. Work was immediately started on the clearing of the



Fig. 8. Interior of Shed.

OCEAN TERMINALS, HALIFAX, NOVA SCOTIA

site and the removal of the old piers, which turned out to be a somewhat tedious work on account of the large number of short stumps of piles which had to be removed by the help of a diver. Meantime the contractors had been preparing their pile-making yard.

The moulding yard measured 800 ft. by 600 ft. wide. A gravel excavating, crushing, and concrete-mixing tower was erected at the east end of the yard, mounted on wheels which ran on tracks across the whole width of the yard. Parallel tracks were laid across the west end of the yard, on which an anchor tower, similarly mounted on wheels, was placed, and a cable way stretched between the two for the conveying of the concrete to the pile moulds.

On top of the main tower, 80 ft. above track level, was placed a winding winch operating a drag-line excavating bucket. This bucket emptied its contents of sand and gravel into a hopper over a grizzly, all stones not passing

through being conveyed to a jaw crusher. The material is then passed over $\frac{1}{4}$ -in. and 1-in. screens to separate the sand from the stone, any stones not passing through the 1-in. screen being elevated to a second crusher.

Chutes from the stone and sand bins led the materials to the mixer platform, where they were measured before passing into the mixer. The concrete mixer was a one-yard cylindrical revolving mixer, which discharged each batch into a bottom dumping bucket which travelled along the aerial ropeway to the pile moulds.

Once the concreting of a pair of piles was commenced, no stop was allowed before they were completed, so that all joints in the concrete were eliminated. The maximum amount of concrete poured in a day of ten hours was 168 cu. yds.

Owing to the weight of the concrete piles, the smallest of which weighed 12 tons and the largest 46,000 lb., the conveying of them to the site of the work presented certain difficulties for which special arrangements had to be made. In order that the piles could be placed accurately in the positions



Fig. 9. North Side of Pier.
OCEAN TERMINALS, HALIFAX, NOVA SCOTIA.

required by the plans, pile leads were hung on a carriage travelling on rollers, which gave it a forward movement of 7 ft., while the leads themselves can be traversed across the face of the carriage 4 ft. on each side of the centre line.

The pile leads, which are built in the form of a three-sided box girder, are 75 ft. in length and weigh about 72,000 lb. Over pulleys on top of the leads pass the cables for handling the piles, the pile guiding-fingers, and the pile hammer. The hammer runs in guides on each side of the leads, which are designed with a switch at the top, so that the hammer can be run into the leads, thus allowing the pile to hang vertically in front.

In order that the brace piles could be driven, the leads are pivoted at the centre of their length, and their lower ends can be carried over 8 ft. to either side of the centre by operating a long threaded horizontal shaft, thus allowing the leads to be canted at the angle required for these piles.

The method followed in placing and driving a pile was as follows: A pile-

driver scow was first placed in position by means of cross ranges, and anchored by lowering her spuds. Straps were placed around the pile to be driven as it lay on the deck of the scow on which it had been brought up from the yard, and the pile lifted horizontally by one of the forward derricks and swung round in front of the driver so that the head of the pile came in front of the pile leads. The swinging of the pile was accomplished by manipulating the forward spuds of the pile-driver scow. The pile fall, having been made fast to the staple projecting out of the head of the pile, was hoisted up, and at the same time the derrick fall was lowered and the pile gradually raised into a vertical position, the straps around the pile being taken off as the strain came off them. The pile leads were then moved as required for the exact centering of the pile. On the pile being lowered it penetrated the overlying soft mud with its own weight. This mud was found to have sufficient stiffness to hold the pile vertically when its depth was about equal to one-third of the length of the pile; as a general rule, however, the piles had to be stayed while the hammer was being adjusted on their heads.

The amount of hammering which the piles required to drive them to refusal varied a great deal. In some places, where the depth of the hardpan was small, the pile would come to a stop after 200 to 300 blows, while in other places they would require nearly 1,800 blows to drive them to rock. The rate of penetration through the hardpan was small, the last foot or so in many cases being penetrated at not more than one-thirtieth of an inch per stroke.

When no machine troubles were met with, an average of ten piles could be set and driven in ten hours. In some weeks, however, the average would slightly exceed twelve per day, and the best day's work was eighteen.

In order to test the bearing power of the piles after driving, load tests were made with a floating load of 180,000 lb. on certain piles. The load, which consisted of two scows of 90,000 lb. weight each, was attached to the ends of a steel girder resting on the head of the pile and made fast at high tide, the pile and load being securely braced to prevent swaying of the pile when under load. On the tide falling, the whole load was carried by the pile. Tests were made on piles which had been driven to different degrees of penetration, with the following results:—

Pile going 1-10th of an inch per blow, settlement under load 1-15th in.

"	"	1-5th	"	"	"	"	"	1-20th	"
"	"	driven to refusal,	"	"	"	"	"	nil.	"

The problem of building forms of the necessary strength and stiffness on the short length of pile projecting above the water was not an easy one, but was successfully solved by the contractors.

The concrete used in the construction of the pier deck and shed was mixed in the proportion of 1 part Portland cement, 2 parts sand, and 4 parts 1-in. crushed gravel. Batch mixers were used throughout, and the concrete was distributed from an elevator tower by chutes. Special care was taken to properly imbed all reinforcing steel, concrete made from a finer crushed stone being used where found necessary.

The surfaces of the floors were finished with a finishing coat of 1 to 2

coarse mortar trowelled to a fairly rough surface. The lower floors were built with a fall towards the railway tracks of 4 ins. in 45 ft.

By the middle of December, 1913, the piles had been driven and the pier floor completed to within 115 ft. of the outer end, at which place the work was suspended until the following spring, and the portion of the pier built was equipped with temporary wooden freight sheds and used throughout the winter by freight steamers. During this time the ability of the pier to withstand lateral blows from large vessels was demonstrated on more than one occasion.

The whole of the concrete work completed by December, 1914, and the lower storey of the shed, is now in full use. The upper storey is at present being fitted up for the temporary examination of immigrants, and was expected to be ready for use in March of this year.

The contract for the construction of the pier and shed has been carried out by the Nova Scotia Construction Company, under the management of Mr. Hamilton Lindsay, assisted by Mr. J. Shepherd Lee, B.Sc. Mr. John Kennedy, M.I.C.E., is Consulting Engineer for the Dominion Government for the work, and was assisted in the design by the writer and by Mr. R. A. Lockerby, B.Sc. The cost of the work, complete, will be about \$1,125,000.00.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

THE CONCRETE INSTITUTE. ECONOMY IN REINFORCED CONCRETE CONSTRUCTION.

By T. A. WATSON, A.M.Inst.C.E.

The following is an abstract from a Paper read at the Fifty-seventh Ordinary General Meeting of the Concrete Institute. A short summary of the discussion which followed is also given.

You will notice that the Paper is supposed to be on "Economy in Reinforced Concrete," which would leave the broader aspect of the economy of reinforced concrete untouched, and would confine the discussion within limits which I did not intend; so with your permission I shall touch on points in the larger sphere, though I may be attempting to make you acquainted with things which you already know.

As you are aware, the London County Council have framed a set of regulations dealing with the construction of buildings entirely in reinforced concrete, specially with a view to the construction of buildings with their external walls of the material, it being supposed that a great economy in building construction would thereby result. I regret to say, however, I have not found that this result has been achieved.

After the new General Post Office had been constructed and Sir Henry Tanner had read a paper on it at the Royal Institute of British Architects, the view was expressed by many architects that a new era had arrived in construction; and so it had, in a way, but not, I think, in the way that most of them seemed to believe.

It was quite true that by adopting reinforced concrete for the General Post Office Sir Henry Tanner had saved a vast amount of public money, some £60,000 in actual construction cost and some £50,000 additional for the value of extra floor space obtained by thin walls, and it was an achievement of which anyone might well be proud; but perhaps the chief value of this building to posterity was the way in which it showed that reinforced concrete could be used more economically still.

We are shortly to have regulations allowing us to build external walls in reinforced concrete. Things have progressed slightly in recent years, and a reinforced concrete framed building with 9-in. or even 14-in. external panel brick walls is even more economical than the original conception of reinforced concrete external walls.

In a way this is disappointing, because it destroys an outlet for the genius of the architect to incorporate in his design the expression of the method of construction in reinforced concrete, leaving him only a skeleton to clothe in brick, stone, marble, tile, or other building material, but to my way of thinking it is true economy.

There is, however, another aspect of walls which I should like to touch on, and that is retaining walls, and particularly retaining walls outside a building. Till quite recently these were built either as gravity retaining walls of sufficient weight and stability in themselves to sustain the earth pressure and superimposed load from pavement and roadway, in which case they were enormously thick at the base, one 20 ft.

high requiring to be about 7 ft. thick, or they were built as a series of vertical arches supported by walls at right angles to them and forming a series of dark dismal vaults.

In both cases the outside of these walls next the earth was lined with asphalt or a vertical damp course of some kind. At the present time reinforced concrete walls are being constructed in many cases without a vertical damp course, which even in damp ground is unnecessary. In passing let me say, however, that in waterlogged strata I personally am of opinion that a vertical damp course is absolutely necessary even with reinforced concrete walls.

But to compare prices and to give gravity walls the best possible chance let us put the average thickness of a wall 20 ft. high at 5 ft. 3 in. thick. In a wall of this thickness there are per yard super—

	s.	d.
672 bricks at 34s. per 1,000	22	10
Extra over 48 bricks for facing at 5s. per 1,000	0	3
$\frac{1}{8}$ -yard cement mortar, 1 : 3	8	9
Labour laying	10	6
Pointing one side only	1	3
Total	43	7

Leaving out the vertical damp course, a reinforced concrete wall, including all counterforts and without any horizontal struts to the main building, can be built for 32s. per yard super. The price is made up as follows:—

	Per foot-run of wall.
	s. d.
Cub. ft. of concrete 27'3, at 1s. 1d.	29 7
Lb. of steel 105, at 1½d.	13 1
Centering 110 ft. super, at 3d.	27 6
	70 2 for 2½ yds. super,

or 32s. per yard super.

After the enormous saving shown above the saving on a reservoir wall seems a mere trifle, but still it is worth having, and it is enhanced by the fact that it is unnecessary to have an asphalt lining on the inside of a reinforced concrete reservoir, as it is not impossible to make a reservoir watertight enough for all practical purposes without this lining. So that even if there were no saving in the cost of reinforced concrete walls against brick for a reservoir, yet there is a saving of 5s. to 6s. per yard super on the lining.

Economy of Pillars.—Take a pillar carrying, say, 200 tons central load, height 16 ft. from floor to floor, ends fixed. A steel stanchion 40 sq. in. in section, weighing 136 lb. per foot-run exclusive of cap base and connections of girders, is required, size 14 in. by 16 in. According to the 1909 Act relating to steel-framed structures this has to be surrounded by 2 in. of concrete (making a total of 18 in. by 20 in.), which latter, to prevent its cracking, has to be reinforced with rabbit wire-netting, or spiral binding of some kind, the total cost of which figures out something like the following:—

	£	s.	d.
Steel stanchion, 16 ft., at 136 lb.; 2,176 at £13 per ton erected...	13	0	0
Concrete surrounding same, 23 cub. ft. at 8d....	0	15	4
Rabbit wire, 80 ft. super at ½d.	0	3	4
Centering, 144 ft. super at 3d.	1	16	0
	£15	14	8

In order to make a saving on this price, and to conform with the regulations proposed, it is necessary to have a column slightly larger in section than the steel one. Suppose, for convenience, we say 24 in. by 24 in.

Allowing a stress of 600 lb. per sq. in. in concrete and 16,000 lb. per sq. in. in steel, the amount of steel required in a column this size is 83 lb. per foot-run including links, binding, etc

Beams.—If we take beams, say one of 25-ft. span of a series at 20 ft. centres, load $2\frac{1}{2}$ cwt. per sq. ft. of floor, including weight of floor. A steel girder consisting of, say, one 14 in. by 6 in. and four 10 in. by $\frac{1}{2}$ in. plates, top and bottom, weight 196 lb. per foot-run, is required. This, when covered with the requisite amount of concrete, gives a depth of beam below floor level of 22 in. and a width of 14 in.; and for the sake of making as near a comparison as possible, take a reinforced concrete beam of the same depth from floor to soffit of beam. A beam of this depth allowing 600 lb. proposed stress in concrete and 16,000 lb. stress in steel requires 110 lb. of steel reinforcement per foot-run, making allowance for lapping of bars and shear members, and as regards weight of steel this compares very favourably with the steel girders.

Floors.—To fill in the floor space 25 ft. by 20 ft., most architects nowadays would probably use some kind of hollow tile reinforced concrete floor, and I am of opinion they would be right; but on the other hand there are some who would use R.S.J. fillers, and the most economical way would then be to divide the 25 ft. up into three bays with two R.S.J. 15-in. by 6-in. at 8-ft. 4-in. centres and use 5-in. by 3-in. by 11-lb. R.S.J. fillers at 3-ft. centres from 15-in. by 6-in. joist to 15-in. by 6-in. R.S.J., and a thickness of floor of 7 in. Comparable with this would be a reinforced concrete slab $5\frac{1}{2}$ in. thick supported by secondary beams at same centres as the 15-in. by 6-in. R.S.J. and supported by the main beams as above. Reinforcement required in $5\frac{1}{2}$ -in. slab amounts to 15 lb. per yard super.

Now, working out the cost in the two cases in a piece of floor 25 ft. by 20 ft., we have for the steel-framed structure £75 14s. 3d., as against £59 14s. 3d. for the reinforced concrete structure,* saving, say, £16 on a piece of floor 25 ft. by 20 ft., or a saving of 10 to 20 per cent. by constructing in reinforced concrete instead of in steel.

Bridges, etc.—It seems hardly necessary to mention that in the construction of bridges, say up to 300 ft. span at least, reinforced concrete is nearly always the most suitable material. There are, of course, exceptions, but the mere question of the cost of maintenance of a steel bridge seems enough to condemn it, whilst a great argument in favour of the reinforced concrete bridge is that it requires no maintenance and becomes stronger as it grows older.

Apart, however, from the question of maintenance, the saving effected in cost of construction of bridges in reinforced concrete over steel bridges is sufficient to justify its adoption, as much as 40 per cent. having been obtained in a bridge of 42 ft. span by 26 ft. wide between parapets.

The question of maintenance is also enough to decide an architect or engineer to choose reinforced concrete for the construction of, say, small water towers, coal bunkers, gasometer tanks, or any similar structure heretofore built in steel and exposed to atmospheric conditions, even if reinforced concrete is not cheaper in the first instance.

With regard to timber structures the same applies, and in addition the relatively greater resistance to destruction by fire makes the advantages of reinforced concrete so apparent that one wonders why timber is employed in the construction of wharfs at all.

Methods for Obtaining Prices.—But to turn to the other side, viz., economy in reinforced concrete, I would like to emphasise a point which in these strenuous days of competition seems to be overlooked, or rather ignored, and that is the enormous amount of waste which occurs through one of the methods adopted by architects to obtain cheap prices for the carrying out of work. An architect prepares the general design of a building and sends sunprints of same to four, five, or even six different firms of specialist designers, who in turn prepare each their reinforced concrete scheme and send out to five or six or more contractors for prices, involving a total of some thirty to forty persons, all of whom spend money and time in tendering, etc. Much valuable time is also wasted which might otherwise be spent in the construction of the building whilst the various schemes and prices are being compared and adjudicated on.

Concrete Surfaces.—During the few years with which I have been connected with reinforced concrete many small problems have been presented to some of which solutions have been found, whilst others appear almost insoluble. One of these latter is the question of leaving the face of reinforced concrete surfaces exposed to view free from board marks. If wrought forms thickened are used, you have in the case of a

* NOTE.—The author gave detailed particulars of how he arrived at these figures.

wall still to put up one side of the wall board by board, and as you fill up with plastic concrete the pressure at the bottom of a 3-ft. lift is greater than at the top, and slight lateral deflection is sure to occur on these separate boards, and marks will appear. I would suggest that architects should endeavour to persuade themselves that it is right for board marks to be there, as they are an expression of the method of construction, the same as the mortar is in brick joints.

Centering.—There is another difficulty with centering, and that is the amount of props required. I would suggest to contractors that great economies might be effected by employing a competent man to design the formwork.

I have often heard it said that when floors repeat time after time in a building centering can be used over and over again and great economy result, whilst if a slight variation is made in the width of the beams and size of columns, extra expense is incurred. Now this latter statement is not always true; props under main and secondary beams must be left whilst succeeding floors are constructed, so in any case fresh beam bottoms have to be used on the higher floors, and no extra beam bottoms are required. In order to facilitate the striking of beam sides it will be advantageous when constructing these to cut them shorter than the distance between the centering to the sides of other beams or columns against which they abut. Fill in the space so left with a loosely fitting strip battened to the side of beam, and put in the usual angle fillet at the junction of beams. When striking the side of the beam take off the last-mentioned battens, remove the loosely fitting strip, and the side of the beam can be easily removed intact. If some similar method be not adopted, and the beam centering is made a good fit, the water in concrete makes the wood swell and jam, with the result that many beam sides are broken or pulled to pieces in being taken down, necessitating the expense of remaking and possibly new material. If the main beams on a higher floor are therefore 1 in. or 2 in. narrower than on the floor below it only means that a slightly wider filling-in piece at the end is required.

Steel Bars.—Another point which makes for economy in reinforced concrete, but has more to do with design, is the number of steel bars that have to be handled. It will be readily understood that it is just as easy to handle a $1\frac{1}{2}$ -in. diameter bar 25 ft. long as it is to handle one that is only 1 in. diameter, as the weight is immaterial; each requires two men to fix it in a beam, and eight bars take twice as long to put in as four, so that from the point of view of economy the four $1\frac{1}{2}$ in., which give the same tensile resistance as the eight 1 in., should be used in preference.

I have only covered some of the ground there is in this question, but I trust that what I have said will be of some use, and if it will promote an interchange of ideas, as I trust it will, some interesting information may be forthcoming in the discussion.

DISCUSSION.

The President observed that what they had to do was to impress the public with the fact of the economy of reinforced concrete, and to impress themselves with economy in reinforced concrete, and then the former economy would be improved by the latter. Compared with steel frame buildings there was generally a considerable saving in time because the reinforcing metal was invariably kept in stock, while for the steelwork the larger beams and stanchions had to be made to order, and very often the designer had not considered which sections could be obtained from stock, and which would have to be waited for until the rolls were next put in. A considerable saving of time might be effected by observing that and utilising stock sections.

He was glad that the author called attention to the waste of energy or money caused by the architect or building owner inviting competitive schemes from a large number of specialists instead of taking counsel from one only, or at any rate from a limited number.

Mr. Percy J. Waldram, F.S.I., said the true economy of any material expressed in monetary terms was of the utmost importance to structural engineers, not only because they were entrusted with the judicious expenditure of their clients' money, but more especially because monetary economy and safety were so often synonymous. To take one example, that of reinforced concrete T-beams for moderate spans and loads, the practical designer gave those a generous proportion of depth to span, $1/10$ to $1/12$, using quite a low percentage of tensile reinforcement, because that gave a much cheaper beam than shallow depths with reinforcements up to or over the economic ratio. In such beams, under working load, only the reliable steel received its working stress, whilst the comparatively unreliable concrete was

stressed very lightly. In the more expensive shallow beams, with, say, 1 per cent. of tensile reinforcement, the concrete was fully stressed under working loads, while the steel got off lightly. The author was to be congratulated upon his frank admission that reinforced concrete did not spell a monetary advantage in every situation. The instances of monetary saving which he gave were at first sight extremely striking, but when they were closely examined they became, to say the least, somewhat less convincing. Alternative estimates were extremely misleading unless they referred to comparable designs similarly priced. It would appear that excellent design in the material with which the author was identified had been compared, doubtless by inadvertence, with design in structural steelwork, which, he thought, might be justly criticised as distinctly and unnecessarily wasteful.

It was worthy of note that the huge block of offices in Kingsway for the Public Trustee was one of the steel frame buildings which, according to the author, were so ruinously uneconomical. One was forced to ask why the interior floors were of the discredited filler and steel beam type if the Office of Works were convinced that reinforced concrete would have saved such huge amounts of the public money of which they were the custodians.

Mr. Percy H. Simco, M.C.I., asked what Mr. Watson meant by saying that the economy of reinforced concrete buildings outside London was greater than inside? Did he mean that outside London he worked to a bigger factor of safety than he did inside London, and, if so, why should not the steelwork man do the same thing?

Mr. Ewart S. Andrews, B.Sc., said the Paper assumed good design on the part of the reinforced concrete designer and rather pointed out the failings of the steelwork, which a good steelwork designer should not make.

Mr. Morgan E. Yeatman, M.A., remarked that, in advocating a smaller number of large rods in preference to a larger number of small rods of the same area, there was one precaution that should be taken: not to use too large rods in a short beam, because they must have sufficient area in proportion to the section of the rods for the strength to be taken up from the surrounding concrete.

Mr. Archibald Scott, A.R.I.B.A., urged that attention should be given to economy in specification.

Mr. W. A. Green, M.A., B.Sc.Eng., did not think a stress of 600 lb. per sq. in. would be allowed in London in a reinforced concrete column. The London County Council would not allow a filler joist 1½ in. wide.

Mr. T. C. Dawson, M.C.I., believed the great trouble in getting reinforced concrete into use in railway-bridge construction was that railway companies were so nervous about the material, and it would be well if the Institute could do something to disabuse the minds of railway engineers of that nervousness. Railway engineers believed that if a reinforced concrete bridge was over-stressed, as many metal bridges were, it would yield very much sooner; they could not place the same reliance upon reinforced concrete.

Mr. B. Flander Etchells, Assoc.M.Inst.C.E., referring to the General Post Office Building which had been cited, pointed out that it was exempt from the constructional requirements of the London Building Act, and many tenders had gone to show that, to conform to the requirements of the old London Building Act of 1894, was still the cheapest method. The Draft Regulations proposed that the stress in a pillar should be 600 lb. per sq. in. on the reinforced concrete, and the stress on the steel in the same pillar should be 9,000 lb. per sq. in.

MR. WATSON'S REPLY.

Mr. Watson, in reply, said that in his Paper he had taken the Regulations governing steel buildings and the proposed Regulations to govern the erection of reinforced concrete buildings in London, and he had compared those two. If one took an isolated case of a building which was exempt from the Building Act no doubt he could show a greater economy on a steel frame than on a reinforced concrete building. Hollow tiled floors were certainly much cheaper than steel joist fillers, and probably than slab reinforced concrete floors, but still they were reinforced concrete floors.

With regard to the question of building inside and outside London, the proposed Regulations governing the construction of reinforced concrete in London were considered by many reinforced concrete people to be somewhat onerous, and they thought that the factor of safety on a reinforced concrete building constructed in London in accordance with the London County Council Regulations became in a few years, not 4, on which it was originally based, but something like 10. The factor of safety of a steel building, whatever it was at the beginning, certainly did not increase with age.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

THE CONCRETE COLUMN BRIDGE AT HUDSON, WISCONSIN.

THE accompanying illustration (Fig. 2) and drawing (Fig. 1) show the construction of the St. Croix River Bridge with reinforced concrete columns at Hudson, Wisconsin. This highway bridge crosses the St. Croix River, and has seven 90-ft. spans, one 50-ft. span, and one 136-ft. riveted steel span truss—all supported on pairs of transverse knee-braced, reinforced concrete pier columns.

There is a special interest attached to these reinforced columns, which are from 4 ft. to 5 ft. in diameter and varying up to about 60 ft. high. They are supported on separate cylindrical piers on foundation piles, and there are also ten 20-ft. viaduct spans supported on single bents and towers of special construction and on pairs of concrete piers.

There is an approach fill located between reinforced concrete walls which have their footings connected by deep, buried transverse struts. The bridge is 1,016 ft. long

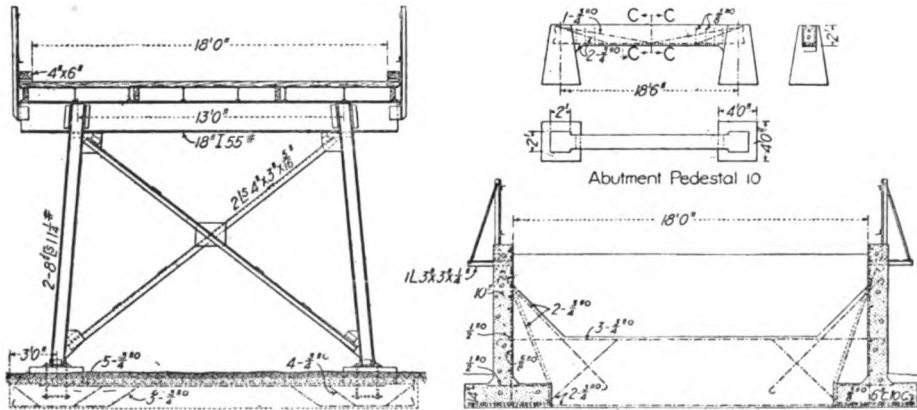


Fig. 1. Construction Details.

THE CONCRETE COLUMN BRIDGE, HUDSON, WISCONSIN.

with a roadway 18 ft. wide in the clear, which has a grade of 7 per cent. and is 48 ft. in the clear above high-water level. The 136 ft. channel span has through trusses to give maximum clearance over the channel, while all the 90-ft. spans are deck spans of ordinary riveted construction and are duplicates, except that the east 90-ft. span cantilevers one panel beyond the pier support at the shore end to carry the 8-in. I-beam roadway stringers at points 10 ft. beyond the pier centres. The through trusses are spaced 20 ft. apart on centres, and the deck trusses are spaced $13\frac{1}{2}$ ft. apart, with the floor-beams at panel points 10 ft. apart, resting on the top chords and cantilevering beyond them. The floor decks have 3-in. transverse planks, supported on wooden stringers 20 ft. long everywhere throughout the length of the bridge except in the through span, which has I-beam stringers.

The main spans have top and bottom lateral diagonals, which in through spans are square rods and in the deck spans $2\frac{1}{2}$ and 3-in. angles. The seven intermediate piers for the eight main spans each consist of a pair of vertical reinforced cylindrical

concrete columns braced together by two vertical transverse webs and supported on separate concrete piers with pile foundations. Piers 6 and 7 supporting the through spans have columns 5 ft. in diameter and 21 ft. apart on centres with bearings on their horizontal upper surfaces for trusses 20 ft. apart.

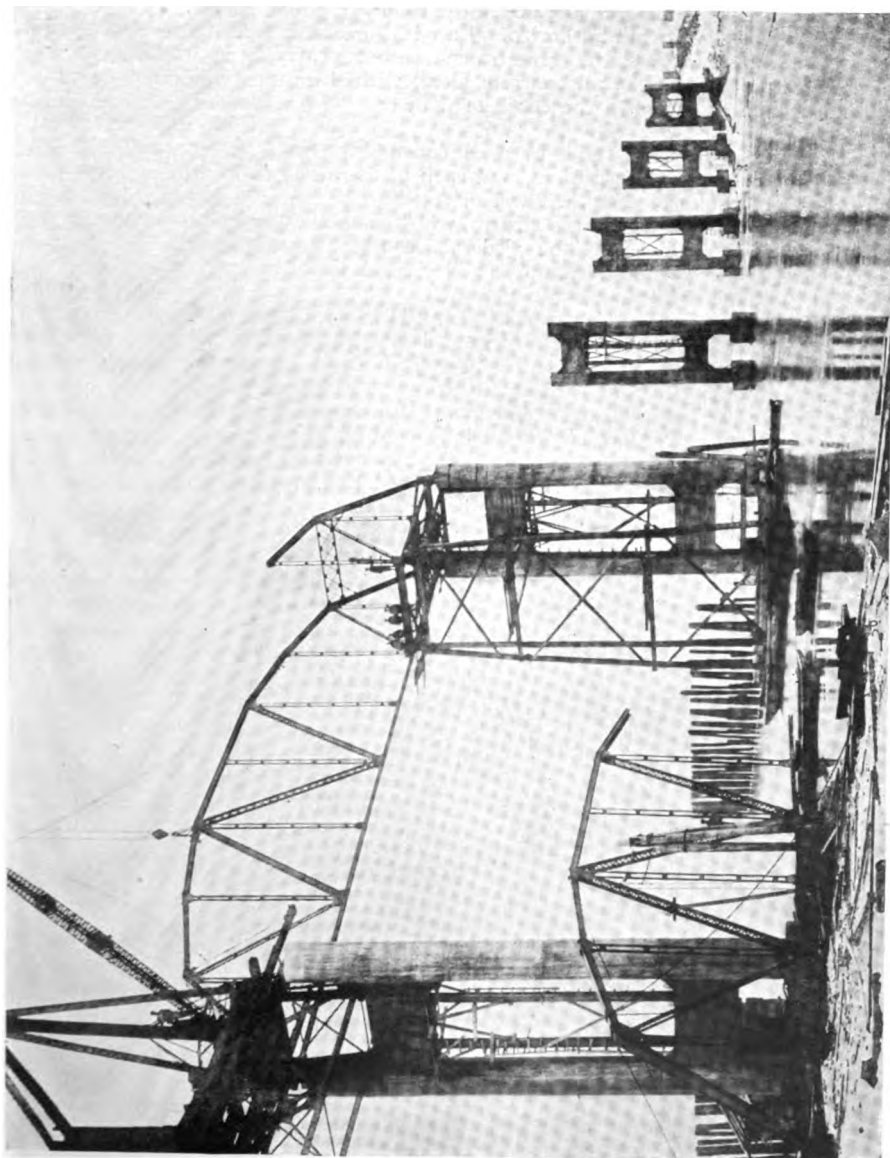


**Fig. 2. View showing Concrete Columns.
THE CONCRETE COLUMN BRIDGE, HUDSON, WISCONSIN**

The upper struts connecting them about 9 ft. below the top act also as girders, with seats 13½ ft. apart on the upper surface to receive the trusses of the deck spans. The pier supporting the deck spans are smaller.

The piers are 8 ft. in diameter and 12 ft. high, and each of these extends down about 7 ft. below the top of the ten foundation piles. The abutment is located at the

top of a high, steep bank and carries one end of a 50-ft. span at one end of the bridge, and consists of a reinforced concrete abutment with wing walls. The other end of the bridge is carried on a solid fill 40 ft. long retained between two reinforced concrete walls with footings extending beyond both faces and connected at the ends of the wall and intermediately every 10 ft. by reinforced concrete transverse struts buried in the fill.

The tops of the walls project just above the roadway grade, serving as curbs, and in them are embedded vertical and horizontal angles for the spandrel posts and braces. Adjacent to the retaining walls the three transverse bents carrying 20-ft. viaduct spans are pairs of rectangular reinforced concrete piers, with their tops connected by 18-in. transverse I-beams embedded in the concrete with the top flanges flush with the tops of the piers.

There are six transverse bents of steel construction between the cylinder pier columns and the concrete viaduct piers, and each of them has two battered posts consisting of a pair of channels with their webs riveted at the upper ends to inclined hitch angles on opposite sides by transverse girders, which cantilever beyond the columns and support the roadway stringers. The girders are 18-in. I-beams with their bottom flanges cut to clear the column channels and riveted to gusset plates engaging the columns and receiving the transverse diagonals.

The columns are seated on short concrete pedestals integral with reinforced concrete transverse members that act as girders and struts connecting the columns below the surface of the ground. The pier columns were built in wooden forms 14 ft. high, raised without removing them as each part of concrete was laid.

The forms for the upper struts were supported intermediately by falsework seated on the lower struts, which were completed long enough in advance to develop girder strength sufficient to carry the wet concrete in the upper struts. The 90-ft. trusses were shipped complete from the bridge shop and hoisted and erected by stiffer derricks moving down grade from end to end of the bridge.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

Report of the Advisory Committee on Rural Cottages.

Eyre & Spottiswoode, East Harding Street, E.C.
96 pp. + iv. Price 1s. 6d.

Contents.—Minutes of Appointment—
Report—Introductory—The Cheap
Cottage—Necessary Accommodation—
General Factors influencing the Cost
of Building—The Requirements of the
Rural Labourer—Design of Labourer's
Cottage—Structure of the Cottage—
Drainage and Water Supply—General
Remarks on the Plans.

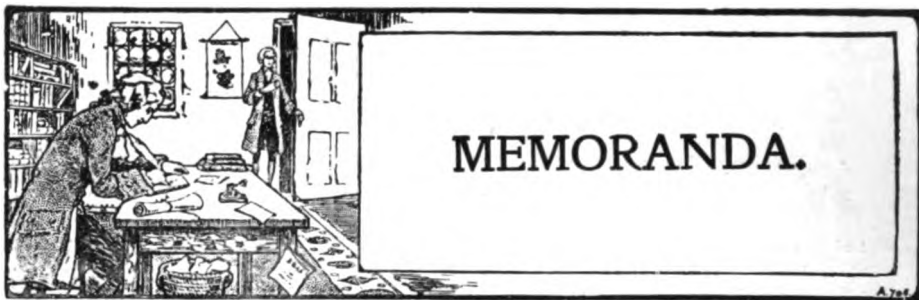
The question of suitable rural cottages of an economical type is of such great importance that a committee was appointed by the President of the Board of Agriculture and Fisheries to consider and advise the Board on plans, models, specifications, and methods of construction for this class of buildings, and the report has just been published. The committee, appointed in November, 1913, was constituted as follows: Mr. Christopher Turnor, Mr. Cecil Harmsworth, M.P., Mr. Raymond Unwin, and Mr. Lawrence Weaver; and Mr. Charles E. Varndell, A.R.I.B.A., was also appointed to the committee in March, 1914.

The general contents of the report are given above, and in addition there are three appendices and twenty-three designs for cottages, together with details of casement and sash windows. Working drawings of any of the designs illustrated can be purchased at prices ranging from 1s. to 2s., and specifications can be obtained for 1d. We do not think this arrangement will be welcomed by architects who practise chiefly in domestic architecture, as it will encourage the speculative builder to dispense with their services; and, in fact, the only thing the architect can do is to purchase the drawings himself and thus save the expense of preparing other designs,

which would cost several pounds to produce.

Although the committee appear to have gone to considerable trouble in their investigations, we cannot say that we consider the report as being at all helpful in solving the problem of cheap and efficient construction, and, in fact, the text appears to state simply those obvious and well-known facts with which every designer is familiar. It is too general in its tone, and does not deal sufficiently with the important details of construction which influence the cost in such a manner that the reader is helped to form definite conclusions as to the methods and materials to employ.

We notice that mention is made of concrete under the sub-heading of "Novel Types of Planning and Construction," and the committee have hopes that this material will prove economical and satisfactory after further experiments, and they also suggest the use of concrete in slabs for out-buildings, as they are satisfied that considerable saving might be effected. In spite of these remarks, however, they do not deal with concrete blocks or quote any examples of their use for the main structure, and no notes are given as to the methods to be adopted when this material is used. Personally, we do not consider the use of concrete blocks as a "novel form of construction," and it must be considered either an oversight or conservatism on the part of the committee that this very practicable and economical material has not been dealt with in a proper manner. We do not suggest that it is the only material to use, or that it will always show a large saving over brickwork, but we do think it has itself proved to be satisfactory and generally more economical, and therefore it constitutes an important factor in cheap cottage construction.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Colliery Electric Lamp Rooms.—In a paper recently read by Mr. Wm. Maurice, M.I.E.E., before the North Staffordshire Mining and Mechanical Engineers, and reported in the *Iron and Coal Trades Review*, the author deals at some length with the question of the design and equipment of colliery and electric lamp rooms. Under the heading "Building Construction" the author stated the following:—

Local custom is usually the guiding factor in the choice of building materials and nothing needs to be said on the point. It is, however, perhaps rather unfortunate that the local bricklayer should often be the arbiter of construction methods. The use of brick footings, lime, mortar, and conventional bondings continues as though Portland cement were non-existent. In 1876, when the Model Bye-Laws were published, Portland cement was very expensive, and not in general use for building operations. Foundations were generally of lime concrete, and brick footings were necessary to protect the lime concrete from crushing. Now the conditions are altered. Cement is so cheap as to supersede lime, and cement concrete being capable of withstanding a greater compressive load than brickwork, the protection of footings is obviously not required, since the bricks themselves would crush before the concrete. Consequently, footings are not provided for the lamp rooms of which constructional details are given in the paper.

For the foundations of the building the trench is dug to the depth required to reach a sound bottom, the labourer undercuts it to an angle of about 6 deg., and then it is filled up almost to ground level with cement concrete, all brick footings being discarded. Bevelled concrete foundations are stronger, cheaper, and superior to the earlier practice in every way.

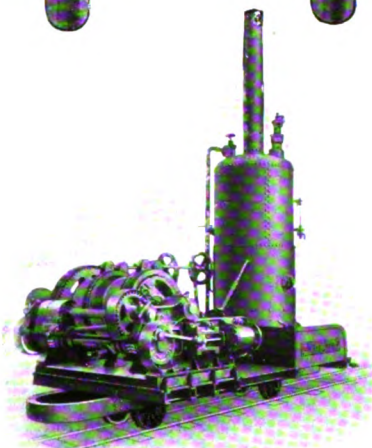
With cement mortar text-book rules for bond can be ignored with impunity, because the conditions are reversed. The matrix is stronger than the aggregate, and the thicker the joints the stronger the wall. The ordinary building brick is porous, absorbing one-sixth to one-seventh its weight of water, and if the walls are only 9 in. thick, with usual methods of bonding, the headers will carry wet right through the walls. For this reason English or Flemish bond is useless for ordinary house construction. In the case of small houses the difficulty is often surmounted by building hollow walls of two 4½-in. thicknesses with iron ties. If the walls are built in cement this will do, but not if they are built in lime mortar. A more effective method, especially for large buildings and where more strength is required, is the grouted wall construction. Two half-brick thicknesses are carried up a height of 10 ft. at a time (if desired), leaving a cavity as with the hollow wall, but only ¾ in. wide. This space is filled in from above with liquid grout, one part Portland cement to one of sand.

Some recent lamp rooms have been built of steel girders filled in with brickwork. Others have been built with flat reinforced concrete roofs.

Kehoe v. Simplex Concrete Piles, Ltd.—This was an action for damages for personal injuries incurred by the plaintiff while working for the defendants on Spicer Brothers' new building in Blackfriars. The plaintiff was crushed between a trolley owing, he alleged, to the unskilfulness of the defendants' foreman. The defendants contended, however, that the accident arose from the plaintiff suddenly and voluntarily leaving a safe position for a dangerous one at the critical moment. On the sitting of the Court, Counsel informed the Judge that the case had been settled, the defendants having agreed to pay the plaintiff £750, including costs. The plaintiff's total claim amounted to something over £1,300.

The BRITISH STEEL PILING CO

A
PLANT
FOR
EVERY
PILE



A
PILE
FOR
EVERY
PURPOSE

THE "ZENITH" STEAM FRICTION WINCH

(as recently supplied to H.M. War Office and elsewhere)

SPECIAL FEATURES

Will lift a 4-ton Pile on one barrel (with Claw Clutch) at 65 feet per minute.

Will lift a 2-ton Hammer on the other barrel (with Heywood Patent Friction Clutch) at 160 feet per minute.

Has Screw-down Brakes to suspend Hammer while pitching the pile, and vice versa.

Has Cook's Patent Governor Valve, which prevents engine racing and automatically regulates amount of steam according to load.

Has Link Motion Reversing Gear.

Is mounted, as required, on flanged or road wheels.

THE MOST COMPACT & EFFICIENT FRICTION
WINCH MADE.

SEND FOR BULLETIN

DOCK HOUSE, Billiter Street, LONDON E.C.

TRADE NOTICES.

New Locks and Latches.—We have received pamphlets from Messrs. Chubb and Son's Lock and Safe Co., Ltd., drawing attention to their patent springless lock and to their new patent front-door latch. The latch, it is claimed, has many advantages, such as a small and neat key, is easily affixed, is suited to either a right-hand or left-hand door, has a dished escutcheon to facilitate entry of key, etc., whilst the patent springless lock is an entirely novel form of springless lock, affording, the manufacturers claim, amongst other advantages, greater security.

Fuller particulars can be obtained from Messrs. Chubb's, 128, Queen Victoria Street, E.C., or at any of their branches.

The Cement Gun.—Our readers will no doubt be glad to learn that the Cement Gun, concerning which one or two special articles have appeared in this Journal some time back, has now a British agent in the person of Mr. R. P. Durham.

The Cement Gun, which is manufactured by the General Cement Gun Co., is a machine for building partition walls, waterproofing and vermin-proofing, trench and conduit lining, repairing concrete and stone work, etc., etc.

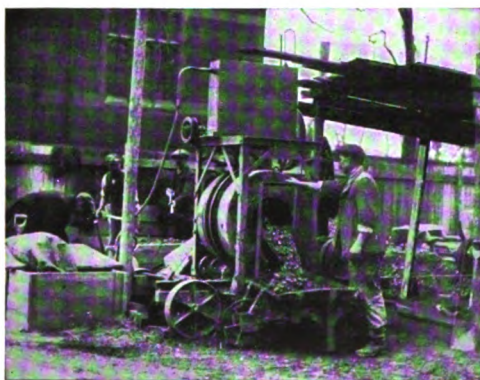
The Cement Gun can either be bought outright, hired, or the Company will contract for the work.

A booklet containing particulars of this invention, together with details of certain work carried out, can be obtained from the British agent, Mr. R. P. Durham, 36 Southampton Street, Strand, London, W.C.

PUBLICATIONS RECEIVED.

Russian Equivalent Tables.—A very useful set of tables has been published by the Central Translations Institute, Ltd. In view of the increasing importance of trade with Russia, the issue of information regarding the somewhat complicated Russian weights and measures, and particularly of equivalent tables permitting of instant conversion of British weights, measures and money into Russian and *vice versa*, is of considerable interest to the trading world. The set of tables compiled and published is stated to cover all ordinary commercial requirements. In addition to money equivalents they include the usual tables for weights and measures.

They can be obtained from the Central Translations Institute, Ltd., Danes Inn House, 265, Strand, W.C. Price 1s. net.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

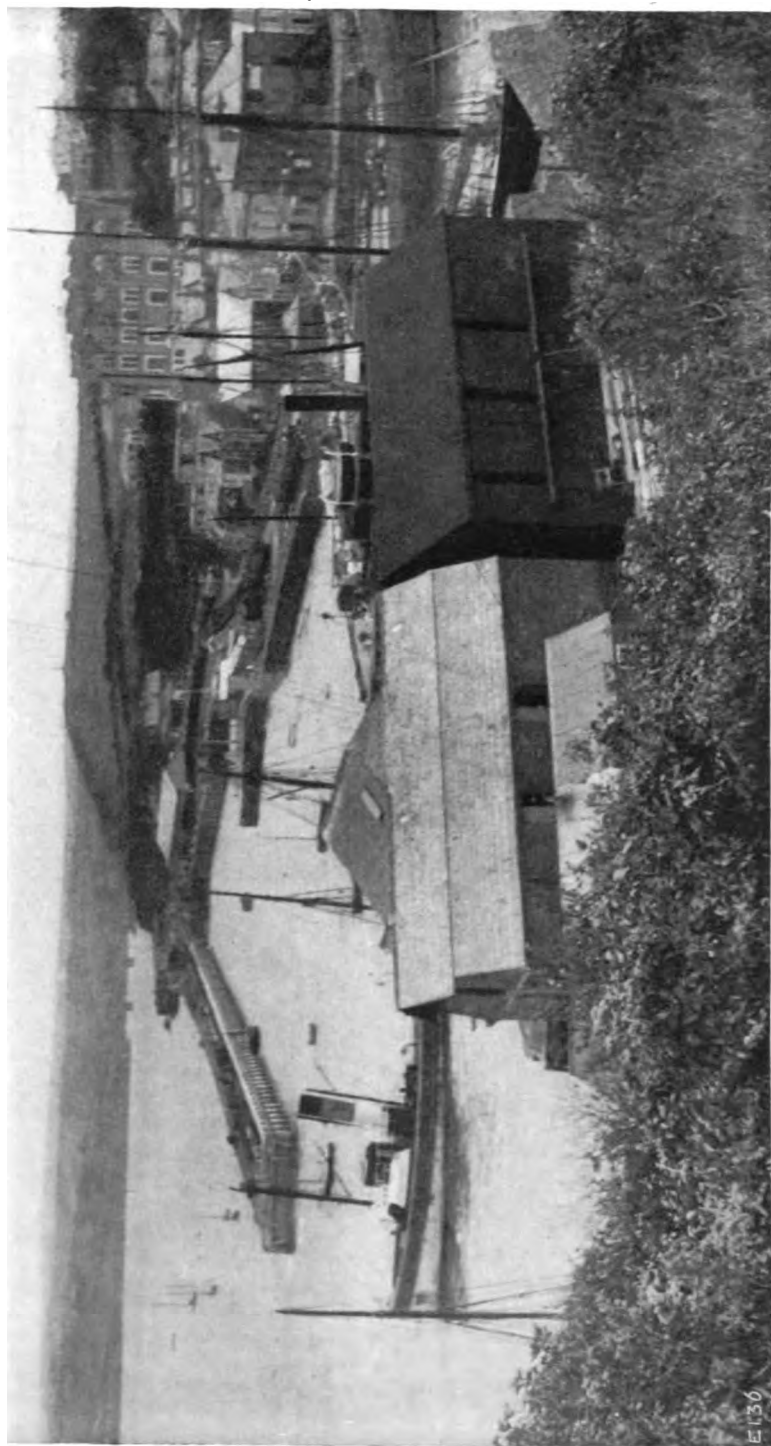
The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



THE NEW HARBOUR AND PIER, PADSTOW, CORNWALL.
(For Description see p. 223)

E136

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X., No. 5.

LONDON, MAY, 1915.

EDITORIAL NOTES.

THE QUESTION OF WIND PRESSURE.

A VERY interesting paper was read by Mr. R. Graham Keevill entitled "Some Notes on Wind Pressure" before the Concrete Institute in March, and an extract is given elsewhere in this issue, together with some notes of the discussion that followed. This subject is of great interest at the present time, when the constructional design of buildings is being developed on highly scientific lines, and it is rather surprising that more attention has not been given to it, especially as the pressure usually assumed is excessive and tends to prohibit economical design. It is absolutely essential that some satisfactory standard should be adopted in order that designers should be in a position to make definite calculations on a basis acceptable to the various authorities and in conformity with the results of actual experiments; and the possibility of individuals adopting pressure varying from 10 lbs. to 56 lbs. per sq. ft. would then be eliminated.

There is no doubt that the members of the Concrete Institute are very dissatisfied with the existing state of affairs, as will be seen upon reading the report of the discussion that took place, and under these circumstances we feel that they should make some effort to deal with the subject of wind pressure and endeavour to clear up any doubtful points both for the sake of the Institute and the engineering profession generally. If the various volumes published by writers of technical matter are consulted, it will be found that a great difference of opinion exists as to the correct pressure to allow and also as to the formula for deriving the pressure value from the velocity. The various bye-laws and regulations in force are just as unsatisfactory, and it is only recently that the pressure adopted in London has been reduced to 20 lbs. per sq. ft. The figure that has been more generally employed in London than any other is 30 lbs. per sq. ft., and although we hardly agree with Mr. P. J. Waldram's remark that a pressure of 20 lbs. per ft. would wipe out half the chimneys in London, we do agree that chimneys constructed in accordance with

the provisions of the London Building Act would overturn under a pressure of 30 lbs. per ft. super, unless the adhesion of the mortar is considered in the calculations, and even then the factor of safety is practically negligible. A case actually occurred where a building owner raised a chimney stack above the prescribed height and refused to put in a tie as requested by the District Surveyor, the latter thereon took out a summons, and before going into Court wished to prepare a diagram to prove that the chimney was dangerous and would overturn. He considered the pressure as 30 lbs. per sq. ft., and found that it would be quite unsafe; but upon preparing a second diagram in which the chimney was limited to the height allowed in the Act he was dismayed to find that this also was unsafe, and his evidence became somewhat unconvincing to a magistrate who possessed no technical knowledge.

It seems very curious that the excessive value of 30 lbs. should have become so generally adopted in face of such examples as these, and it can only be accounted for by the lack of definite investigations and the uncertainty which appears to exist in the minds of all engineers as to what is a reasonable velocity to allow for, and how the pressure can be calculated from such velocity.

We have really made practically no progress in this subject since the time of Sir Isaac Newton, as, according to his law $p = 0.0027 V^2$, and at the present day this value is still adopted by many writers. According to Rankine $p = 0.0054 V^2$, and it is curious to note that with all the advances made in recent years the two values of Newton and Rankine are still both adopted.

There are two modern American text books, each of which is regarded as an authority that can be given as an example. Marburg, in his "Framed Structures and Girders", under the heading of wind pressure, gives the value as $p = 0.0027 V^2$, while Burr and Falk, in the "Design and Construction of Metallic Bridges," state that $p = 0.0054 V^2$, and furthermore the authors in both cases start with the same assumptions.

It is quite clear, therefore, that even if a certain known velocity is considered there will still be a great difference of opinion as to the pressure that would result therefrom.

The experiments carried out by Dr. Stanton in 1903 resulted in his obtaining the value of 0.0027, and there is no reason why this should not be definitely adopted until further experiments can be carried out to confirm it, or produce a different value of undoubted accuracy.

During the discussion at the Concrete Institute some interesting remarks were made by Mr. E. Gold, of the Meteorological Office, and he stated that the greatest velocity ever experienced in London occurred in January, 1881, when it reached 43 miles per hour, and this has never been exceeded since. The nearest approach to it was in March, 1913, when a velocity of nearly 40 miles per hour was reached, and these facts should be sufficient to show how excessive are the pressure and velocities now generally considered. If the velocity in London is considered as 50 miles per hour for the purpose of calculation, this according to the formula $p = 0.0027 V^2$ would only give a pressure of $6\frac{3}{4}$ lbs. per sq. ft., which is somewhat different to the amount provided for in the regulations in force, which call for excessive strength and involve waste of material.

Of course the effect of suction pressure should be dealt with, and this will

materially affect the design of roof trusses and similar members where the pressure on the windward side only is generally considered.

The engineering student, upon commencing the study of theoretical construction, would find that he could not take any definite value or method in dealing with wind pressure with the certainty that he was proceeding on satisfactory lines, or even that he was following the methods adopted by the majority of designers, as so many different values are given in text-books, and each authority consulted but leads to greater confusion. Such a state of affairs is very unsatisfactory, and results in training which is inconsistent, thus producing engineers each working on a different basis and each with a different opinion, as is evidenced by the various views of the members taking part in the discussion before referred to. As it is not possible for the individual to conduct experiments on a satisfactory scale to derive empirical formulæ for himself, he naturally looks to the Institute of which he is a member to put him in possession of information of a reliable character. As the pioneers of scientific construction in this country the Concrete Institute should take up the question of wind pressure and its relation to the design of buildings, and endeavour to get a standard adopted which is more in conformity with the requisite allowance, thus promoting economical design and construction.

OBITUARY NOTICE.

The late Mr. Edmond Coignet.

It is with the greatest possible regret that we have to record the death of Mr. Edmond Coignet, which occurred on the 29th March last, in Paris, after a serious illness. He will be remembered by all as one of the pioneers in reinforced concrete. Mr. Edmond Coignet was well known in London personally, owing to his frequent visits to this country, where he endeared himself to a large circle of friends. He had a personality which compelled many admirers, and this was perhaps largely due to the fact that, apart from being an engineer of distinction, his knowledge of the world and his wide enterprises outside reinforced concrete construction—i.e., as a banker in finance, as a great contractor, as a leader in great industrial schemes—gave him that wide outlook which prevented his being too exclusively interested in the speciality with which his name is largely associated.

The late Mr. Coignet studied engineering at the "Ecole Centrale" of Paris. In 1879 he was created Officer of the "Legion d'Honneur," and he obtained all the highest awards for his well-known system of reinforced concrete. He was gifted with an extraordinary amount of energy, which he applied to a large number of branches of human activity. In fact, he was not only an eminent engineer for works in reinforced concrete, but he was also thoroughly versed in all other branches of engineering. He constructed a large number of railways and electric tramways in France, in South America, and latterly in Italy. A few years ago he entered into a contract for the construction of the Port of Bahia, in South America. This work, the cost of which amounts to several million sterling, is not yet completed.

During recent years his advice was sought by the directors of the "Ecole Centrale" in Paris concerning improvements in their curriculum. He was

president of the "Chambre Syndicale des Constructeurs en Ciment Armé." He was also a member of the Chamber of Commerce of Paris.

Mr. Coignet was not only well known as an inventor and a man of great practical experience, but he possessed also a very considerable knowledge as a theorist.

He was awarded the "Baude" Gold Medal for having defined in a remarkably clear manner the basis of the calculation of reinforced concrete in his celebrated communication to the "Société des Ingenieurs Civils de France," and his theories were confirmed in 1906 by the works of the "Commission Ministerielle du Ciment Armé."

In 1892 he persuaded the engineers of the Paris County Council to adopt his system for the construction of a large sewer at Achères, near Paris, which brought about a considerable saving on the original design in heavy concrete masonry. This work, which was visited a few years ago by the Concrete Institute, of which Mr. Coignet was an honorary member, was the first large sewer constructed in this material.

Mr. Coignet's name, however, became universally known on account of the monumental waterworks known as the "Chateau d'Eau," which he designed and constructed for the Paris Exhibition of 1900, and for which he was awarded the "Grand Prix" and a Gold Medal.

Amongst his numerous patents concerning reinforced concrete we would mention, as having an exceptional importance, his patents for piles, the application of which in this country brought about a celebrated patent action, which lasted several years, and in which Mr. Coignet was finally successful before the House of Lords.

Mr. Coignet's method of construction has been applied to a large number of important works in this country; amongst others, we may mention, several large buildings for H.M. Office of Works.

He was also the chairman of a bank in Paris, which he created for the special purpose of dealing with industrial enterprises.

Reinforced concrete loses a really great man by his death.



Fig. 1.

REINFORCED CONCRETE PIER AT PADSTOW.

By Sir G. CROYDON MARKS, M.P.,
Assoc.M.Inst.C.E., M.I.M.E.

The following article on the Padstow Pier will probably be of interest to engineers, especially as there were many difficulties to overcome in its construction. We would add that this important structure is one of the latest executed in this country on the Coignet System.—ED.

THE increasing trade of the port of Padstow, on the North-East Coast of Cornwall, combined with the developments of the Fishing Industry on that coast, made it incumbent upon the Harbour Commissioners to provide for further accommodation and to so introduce it as would permit of no interruption of their existing trade or any decrease of the accommodation provided for the vessels regularly using the harbour.

The Harbour Commissioners consulted the author, who suggested two schemes for securing the increased accommodation—one for a new pier running from the outer arm or jetty of the old harbour, while the other was for an entirely new enclosed water area with provision for a wet dock and gates extension that might hereafter be constructed, adjacent to the sidings and wharf of the London and South-Western Railway Company. The Harbour Commissioners decided to adopt their consulting engineer's latter proposal, and obtained the Act of Parliament for authorising such new harbour work.

The surveys and plans for the Parliamentary powers were prepared by the author and Mr. Edward C. R. Marks, A.M.I.C.E., M.I.M.E.; and upon the working plans and contract being approved by the Harbour Department

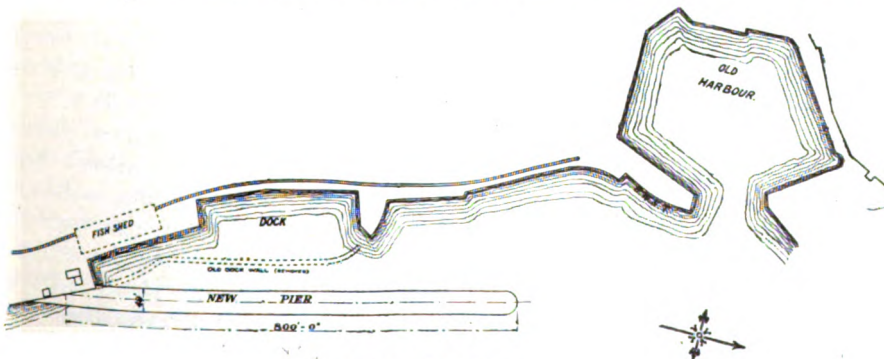


Fig. 2. Site plan.
PADSTOW PIER, CORNWALL.

of the Board of Trade, a substantial Government contribution towards the cost of the construction was promised and in due course paid. The Parliamentary plans provided for a new pier of a length of 800 ft. and a width of 40 ft., having two separate lines of rails, with crossings laid on the surface standing about 25 ft. 6 in. above the level of low-water spring tide.

After the Parliamentary powers were obtained by the Act of 1910, the engineers prepared working drawings for a pier to be constructed throughout with concrete walls and having ordinary rammed filling between them. Tenders upon these original plans were obtained from a number of well-known firms of contractors experienced in such work, but the author, when preparing such plans, expressed his opinion to the Harbour Commissioners that from his acquaintance with constructional works that had been carried out in reinforced concrete it would undoubtedly be less costly and equally satisfactory to employ reinforced concrete instead of ordinary massed concrete for the main outer walls. This suggestion was accepted by the Harbour Commissioners, and six firms, selected from those who had tendered for the original massed concrete scheme were asked to submit tenders upon new plans for reinforced concrete construction.

After very exhaustive enquiries and careful investigation, the engineers decided to co-operate with Messrs. Edmond Coignet, of Victoria Street, Westminster, the well-known

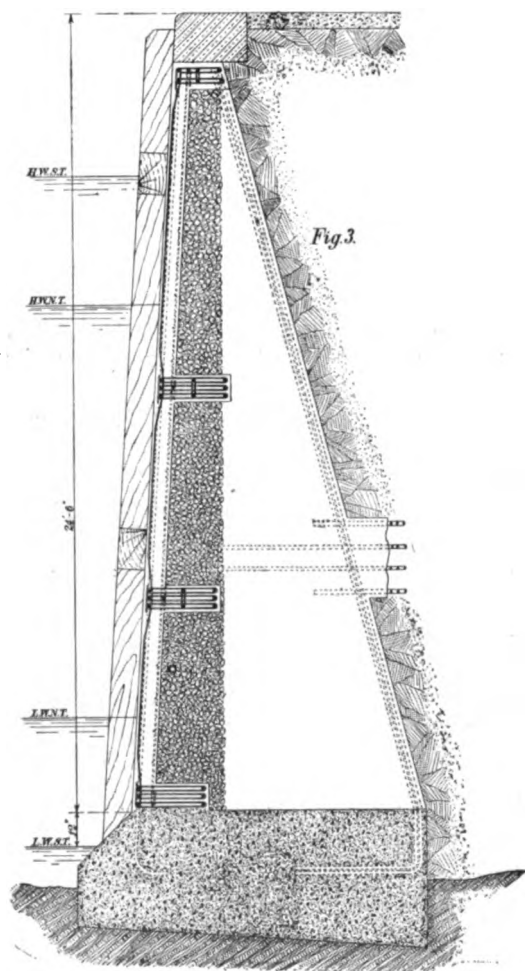


Fig. 3. Section of Inner Wall.
PADSTOW PIER, CORNWALL.

specialists in reinforced concrete construction, in adapting and converting their original plans for massed concrete work into new plans for reinforced concrete on the Coignet system, as adopted by that firm for other works in different parts of the world. The revised plans and specifications were submitted to tender, with the result that the tender of Mr. A. Carkeek, of Redruth, was accepted, and this tender, while being not greatly below other tenders upon

the same specification, was some 20 per cent. lower than the corresponding tender for the ordinary concrete construction upon the original plans.

The difficulties connected with the construction of the pier were very great, owing to exceptionally stormy weather and the fact that the water never left a large portion of the heavy foundation and excavation, which consequently had to be put in with the aid of sheet piling and coffer dams. The tide, too, which was, when in, some 25 ft. above the foundation, rendered it necessary that all work undertaken should be by night and day shifts of men working only between the tides. The work was commenced in July, 1911, and completed in June, 1914, the contractor's period of maintenance provided for by the contract expiring in December, 1914.

From the plans illustrated herein it will be seen that the pier consists of two outer walls constructed in reinforced concrete, having cross-tie beams also



Fig. 4. Erecting the Reinforcements.
PADSTOW PIER, CORNWALL.

of reinforced concrete running from one wall to the other, and having also a number of inside buttresses for the entire length of the walls. The lower foundation members were of massed concrete, into which the metal reinforcements were well tied and embedded, while the concrete surrounding and carrying the reinforced members of the buttresses was also incorporated with and deposited into such solid or massed foundations which rested and also keyed on the solid rock of the harbour bed.

The massed or solid concrete foundations were provided with toes which engaged with the rock. It was decided, in order to facilitate the work of construction, particularly on account of the varying depths of water met with, that massed concrete should be employed throughout for the lower foundation, and that such should be of a mixture of four to one wherever the depth of the rock was greater than low-water at spring tides, while the height of the

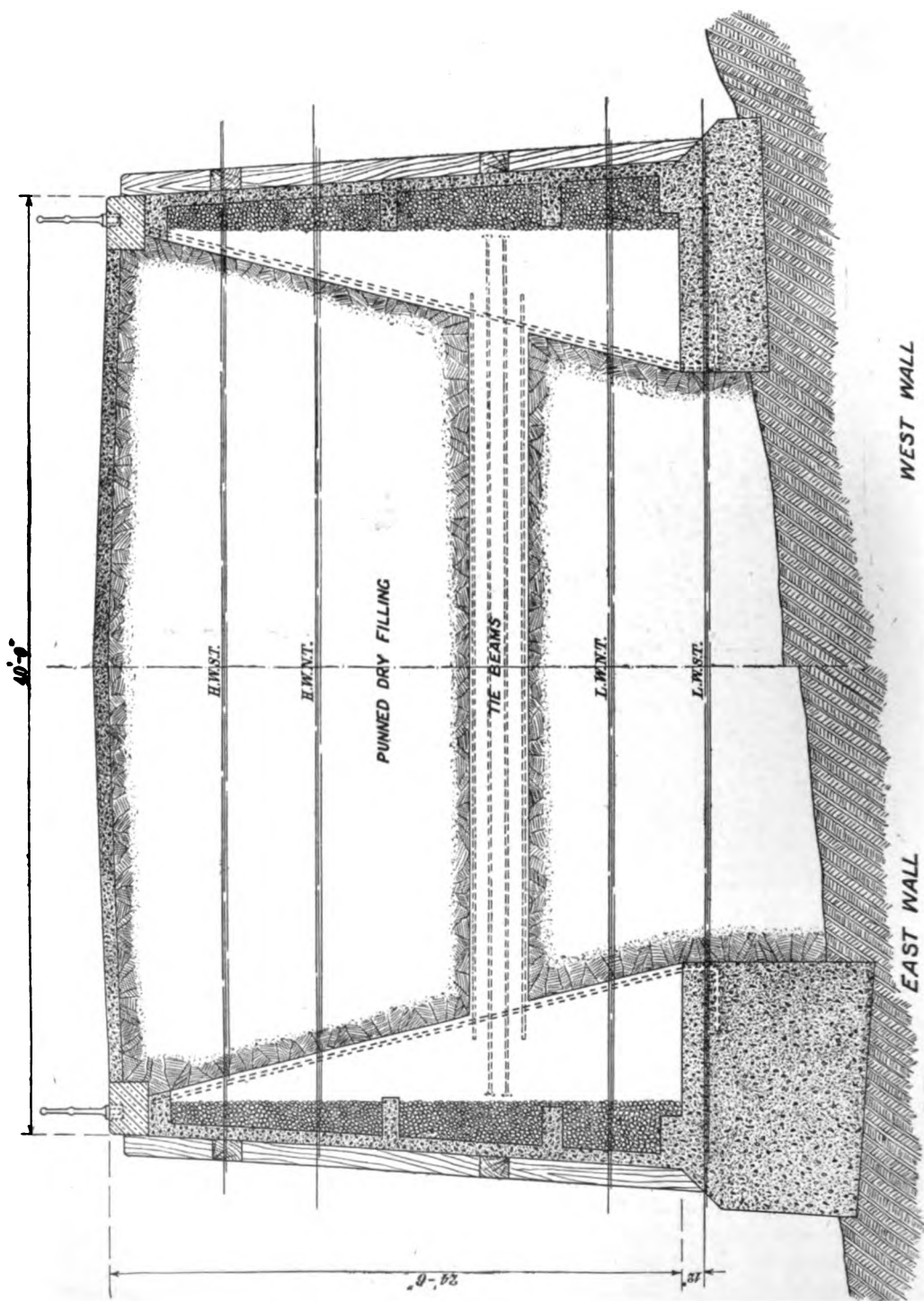


FIG. 5. Cross Section,
PADSTON PIER, CORNWALL.

massed concrete was made to vary with the irregular level of the rock. In every case, however, the top of the massed foundations was carried a foot above low-water to enable the contractors to construct all the reinforced concrete work above low-water, the engineers being of opinion that it would be extremely difficult to properly provide for the construction of reinforced concrete below the level of low tide.

The reinforced concrete walls were provided with a continuous meshwork of steel bars of $\frac{3}{8}$ -in. diameter, while the wall itself was divided into three

panels supported by continuous horizontal reinforced concrete beams 10 in. high and 20 in. wide; the buttresses have a width of 9 ft. at the base where connecting to the massed concrete, and tapered gradually to the top of the wall. The entire main bars of the beams and the buttresses are bound together by means of metal cross stirrups or double embracing loops to offer resistance to shearing stress.

Owing to the rising and falling of the tide amounting to about 20 ft., the walls were subjected when standing alone to very severe stress, of a character even greater in degree and extent to that which is likely to arise under the normal or finished conditions of the pier. To provide for thus



Fig. 6. Massed Foundation and Reinforced Walls.
PADSTOW PIER, CORNWALL.

initially and thoroughly testing the strength, it became necessary to make very substantial provisions for the self-contained stability of the work, both in the case of low tide and of high tide, quite apart from the ordinary working conditions that had to be provided for the load that would need to be carried and the stress induced by trains travelling upon the rails laid upon the surface of the material held between the walls.

The beams, panels and buttresses, and all the component parts of the work were liberally calculated for resisting inward and outward thrusts, while the concrete was composed of a rich mixture in order to prevent the possibility of the sea-water attacking the reinforcements when in position. To prevent cracks arising from expansion or contraction, expansion joints were provided at intervals by forming two buttresses, so that they were adjoining each other with a space of one inch between them. Suitable weep-holes were provided in various parts of the walls to prevent accumulation of water within.

The two sectional elevations show the general arrangement of the bars disposed in the reinforced concrete. Wherever the foot of the wall and

buttresses rest directly on the rock, a longitudinal footing in reinforced concrete is provided consisting of three heavy inverted beams and a slab capable of supporting the weight of the filling, with the object of fixing the buttresses in such a way as would resist any stress towards overturning the wall from the internal pressure at low tide. In positions where the reinforced concrete wall and the buttresses rest upon mass concrete, the reinforcements were anchored and punned directly into such. A certain number of joints were provided to form a good connection between the mass concrete and the reinforced concrete work. The outer surface of the walls are protected by pitch-pine fenders, 10 ft. apart, arranged vertically, and having also a double row of longitudinal members between them. The face of the reinforced concrete walls has a batter of one in fifteen, and the upper surface is provided with Cornish granite coping running the entire length, while the dry filling at the back of the walls is of hand-packed rubble to a depth of about 2 ft., the intervening space being filled with well-punned spoil from adjacent quarries laid in successive layers of about 1 ft. Chains diagonally disposed with anchorage beams were let into the ground, heavy links and rings being provided above the surface for the use of mooring attachments for the shipping.

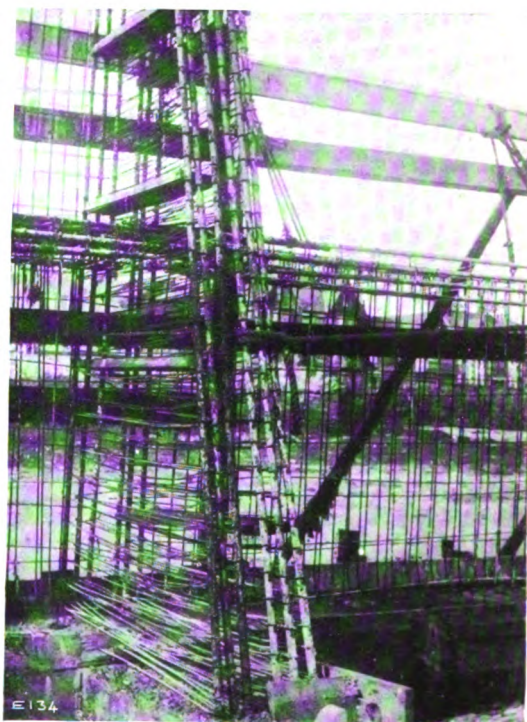


Fig. 7. Reinforced Skeleton Beams.
PADSTOW PIER, CORNWALL.

The whole of the granite that was employed was obtained from the Cornwall Co-operative Company, St. Breward.

The Portland cement was sufficiently fine to pass through a sieve for 2,500 meshes per sq. in. without leaving any residue, and through a sieve of 5,625 meshes per sq. in. without leaving more than 15 per cent. residue. Sample test briquettes placed in water twenty-four hours after gauging and remaining there until tested were required to bear after 7 days 400 lb. per sq. in., and after 28 days 500 lb. per sq. in.

Sample cubes made by the contractor at the beginning of every fresh piece of work were made and tested by Messrs. Kirkcaldy, who also tested the cement before such was permitted to be used.

Fresh water alone was employed for the mixing of the concrete, while the centering and shuttering were kept long enough in position to enable the concrete to become sufficiently hard to maintain itself.

All the concrete was well rammed, so as to fill the casings and surround the bars, and was deposited gradually in thin layers; when it was necessary to suspend the work of concreting, a rough facing was left in order to form a good key for the new concrete while the old face was well cleaned with water before a new deposit was made.

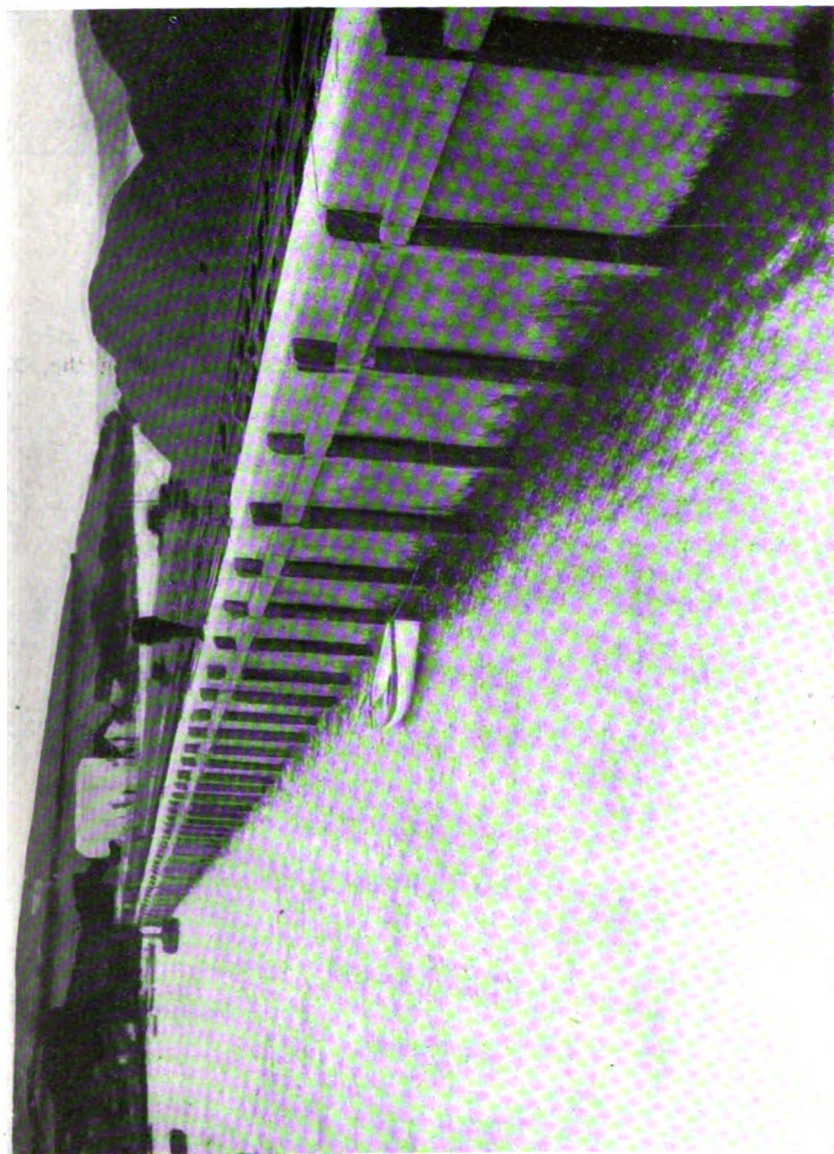
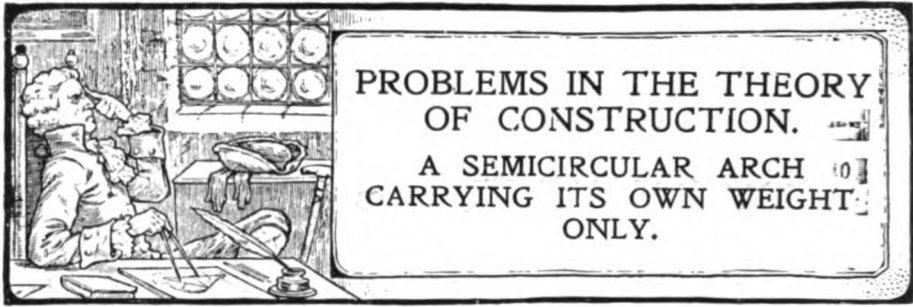


Fig. 8. New Harbour and finished Pier.
PADSTOW PIER, CORNWALL.

The working detail drawings of the reinforced construction throughout designed on the Coignet system were prepared under the writer's general direction by Messrs. Edmond Coignet, Limited, of Victoria Street, Westminster, the resident engineers being Messrs. C. Rickard and G. H. Reed and W. E. S. Toker, under the general personal supervision of the writer and Mr. E. C. R. Marks.



By EWART S. ANDREWS, B.Sc.(Eng.).

The following short article will probably be of interest to engineers.—ED.

IN the discussion following the writer's Paper before the Concrete Institute upon "Some Modern Methods of Arch Calculation," Professor Adams asked if a mathematical solution could be given to the problem of finding the line of pressure for a semi-circular arch carrying its own load only.

Regarding the ends of the arch as hinged, the following solution is obtained of the problem. If an isolated load F (Fig. 1), acts at a point P on an arch APD , the horizontal thrust at the supports is given by the elastic theory of arches by the formula

$$H = \frac{F \cos^2 \phi^*}{\pi}$$

Now let w be the weight per unit length of the arch; then the weight of a short arc subtending an angle $d\phi$ at the centre will be $w \times \text{arc} = w R d\phi$. $\therefore$ Horizontal thrust contributed by this arc

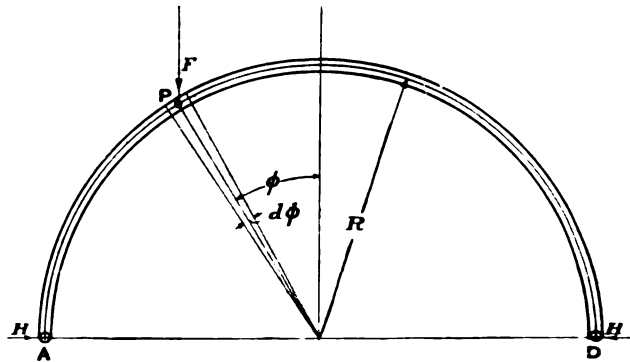


Fig. 1.

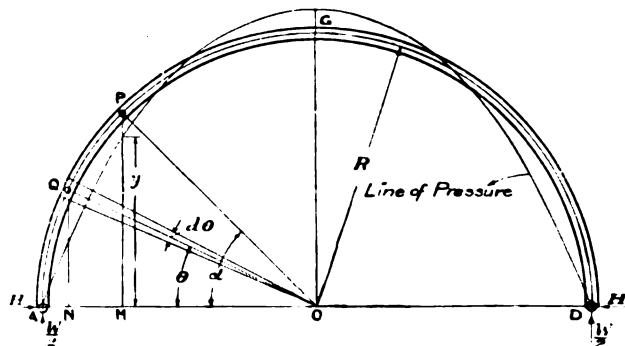


Fig. 2.

A SEMICIRCULAR ARCH CARRYING ITS OWN WEIGHT ONLY.

* This formula is proved in the writer's "Further Problems in the Theory and Design of Structures" (Chapman & Hall, Ltd.), p. 136

$$\begin{aligned}
 &= dH = \frac{w R \cos^2 \phi d\phi}{\pi} \\
 \therefore H &= \int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \frac{w R \cos^2 \phi d\phi}{\pi} \\
 &= \frac{w R}{\pi} \int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \cos^2 \phi d\phi \\
 &= \frac{w R}{\pi} \int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \frac{(1 + \cos 2\phi)}{2} d\phi \\
 &= \frac{w R}{\pi} \left[\phi + \frac{\sin 2\phi}{4} \right]_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \\
 &= \frac{w R}{\pi} \cdot \frac{\pi}{2} = \frac{w R}{2}
 \end{aligned}$$

The total weight of the arch = $W = w \times \text{circumference}$

$$= w \cdot \pi R$$

$$\therefore H = \frac{W}{2\pi} = .1592 W \quad (1)$$

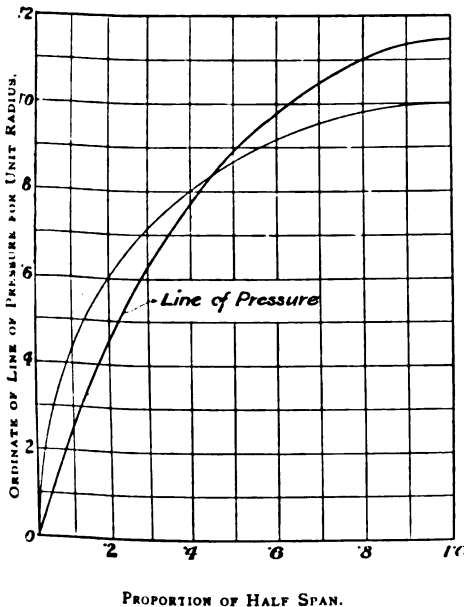


Fig. 3.

To calculate the ordinate y , Fig. 2, of the line of pressure at the point P we first calculate the "free bending moment" B_0 at P .

An elemental arc at Q contributes a bending moment at P equal to $w R \cdot d\theta \times NM$

$$= w R d\theta (R \cos \theta - R \cos a)$$

$$= w R^2 (\cos \theta - \cos a) d\theta$$

$\therefore$ Total moment at P due to weight of portion from A to P =

$$\int_0^a w R^2 (\cos \theta - \cos a) d\theta$$

$$= w R^2 \left[\sin \theta - \theta \cos a \right]_0^a$$

$$= w R^2 (\sin a - a \cos a)$$

$\therefore B_0 = \frac{W}{2} \cdot AN$ -- Moment of weight of arc AP about P

$$= \frac{WR}{2} (1 - \cos a) - w R^2 (\sin a - a \cos a)$$

$$= \frac{WR}{2} (1 - \cos a) - \frac{WR}{\pi} (\sin a - a \cos a) \quad (2)$$

Now $y = \frac{B_0}{H} = \frac{B_0}{W} \cdot \frac{2}{\pi} \left(\text{from (1)} \right)$

$$= R \left\{ \pi (1 - \cos a) - 2 (\sin a - a \cos a) \right\} \quad (3)$$

In using this formula a must be in radians.

This gives the following values of $\frac{y}{R}$ for various values of a

a in degrees.	30	45	60	75	90
$\frac{y}{R}$	.328	.617	.886	1.074	1.142

Fig. 3 has been drawn to facilitate the use of the formula. The radius has been taken as unity, so that for any given case we have only to multiply by the radius the ordinates of the line of pressure given in the figure to obtain the ordinates of the required line of pressure.

If instead of drawing the line of pressure we wish to calculate the actual bending moment B at any point P we proceed as follows:—

$$\begin{aligned} B &= B_0 - H \cdot P M \\ &= B_0 - \frac{W}{2\pi} \cdot R \sin a \\ &= \frac{WR}{2} (1 - \cos a) - \frac{WR}{\pi} (\sin a - a \cos a) - \frac{WR \sin a}{2\pi} \quad [\text{from (2)}] \\ &= WR \left[\frac{(1 - \cos a)}{2} - \frac{(1.5 \sin a - a \cos a)}{\pi} \right] \quad (4) \end{aligned}$$

At the crown where $a = 90^\circ$ this gives

$$B = \frac{WR(\pi - 3)}{2\pi} = .0225 WR$$

or putting $L = 2R$

$$B = \frac{WL}{89} \text{ nearly.}$$



Fig. 1. The Old Quay.

REINFORCED
CONCRETE
QUAY WALL,
SOUTHAMPTON

We are indebted for the following particulars and illustrations to Mr. J. A. Crowther, Borough Engineer, Southampton.—ED.

THE Quay is being constructed on the tidal portion of the River Itchen, for the Corporation of Southampton, by Concrete Piling, Ltd., of 43, Broadway Court, Victoria Street, Westminster, to the design of the Borough Engineer, Mr. J. A. Crowther, the contract price being £3,864.

It has to support a wall of earth varying in height from 16 ft. to 20 ft. and also to carry a surcharge load of road granite to a height of 6 ft. Vessels of 800 tons can berth at this quay at high tide when a draft of 14 ft. is available; it is also used to berth the hoppers which carry sludge and refuse out to sea.

The design, primarily, consists of a double row of king piles, spaced 12 ft. from front to back and at 8-ft. centres longitudinally, which are connected by a deck beam at the head and by a tie beam at a lower level. At the back of the front king piles, sheet piles are driven until a good set is obtained, and then the heads are broken down and a waling is formed on them, from which an inclined slab, 17 ft. in length, is formed at an angle of 55° —this being supported at the top by the back upright. At this point, also, the slab is connected to the decking, which is 9 in. thick and runs the whole length of the quay, connecting the heads of all uprights and forming a horizontal girder, 12 ft. wide, giving great rigidity.

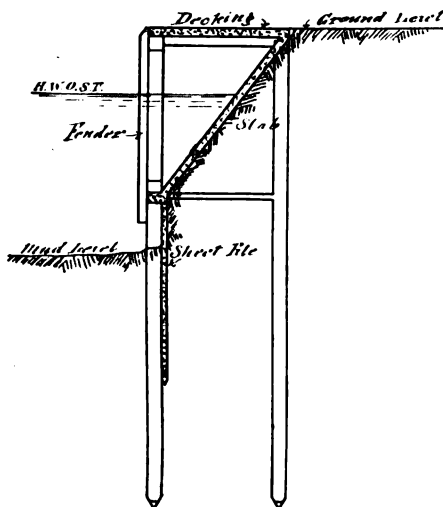


Fig. 2. Showing Type of Construction.
REINFORCED CONCRETE QUAY WALL, SOUTHAMPTON.

On the front section, haunchings are formed at the junction of all vertical and horizontal members, but in the back section they are omitted. When completed, the quay will consist of forty-three 8-ft. bays, with



Fig. 3 Pile Drivers for Sheet and King Piles.

a short return end formed of a single line of 8-in. sheet piles, with king piles at 12-ft. centres, anchored at the top by 1½-in. steel land ties.

The decking will be finished with a wearing surface of granolithic paving and fenders will be bolted to the front line of piles, while mooring rings with chains will be anchored to blocks of concrete some distance back from the quay.

The whole of the piles were made upon the site; the king piles, being 30 ft. long and 15 in. square, were reinforced with No. 4 1½-in. mild steel bars, and in addition with No. 4 1½-in. bars for a length of 14 ft. to give greater strength against bending at mud level. The sheet piles were made in 14-ft. and 18-ft. lengths

of 18-in. by 6-in. sections, and reinforced with No. 6 ¾-in. bars.

Upon driving the king piles, it was found that a good final set was obtained



Fig. 4. King Piles on site.

REINFORCED CONCRETE QUAY WALL, SOUTHAMPTON.

at roughly 22 ft. below mud level. With a steam driven monkey of $2\frac{1}{2}$ tons, delivering blows at the rate of thirty per minute, a set of 4 in. in twenty-four

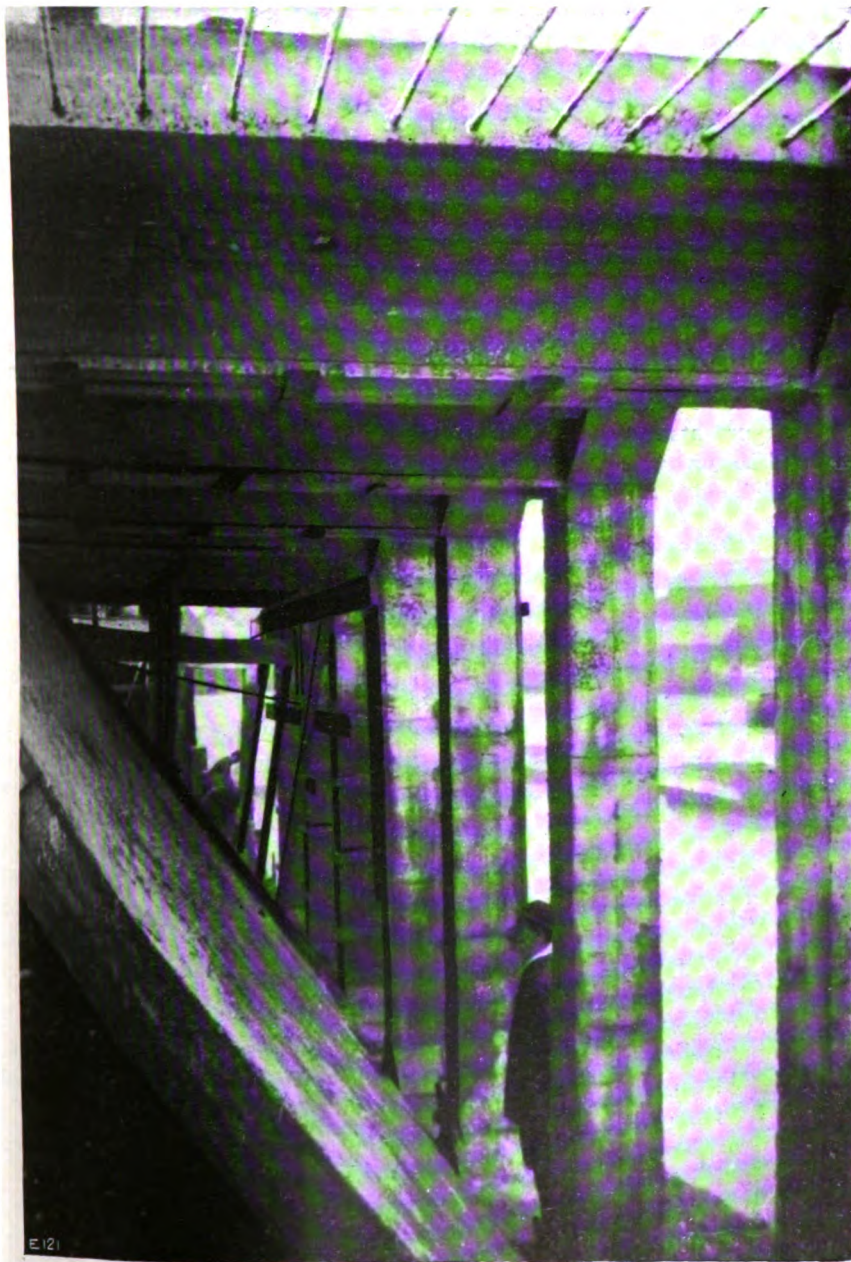


Fig. 5. End View, showing Front Uprights, Inclined Slab, and Decking.
REINFORCED CONCRETE QUAY WALL, SOUTHAMPTON.

blows is obtained, this being considered satisfactory. For the sheet piles a 1-ton monkey is employed, giving a $\frac{1}{2}$ -in. set for a drop of 6 ft.

All piles are broken down, the king piles 24 in., and sheet piles 6 in. to -2'00 O.D. for the formation of the 21-in. by 9-in. waling and the 15-in. by 6-in. tie to back pile. The waling is reinforced with No. 6 $\frac{1}{4}$ -in. bars, and the tie with No. 4 $\frac{1}{8}$ -in. bars.

The inclined slab is subjected to very little earth pressure, as the natural slope of the clinker filling behind the quay is roughly 45° ; but on account of its great length it is doubly reinforced with $\frac{1}{8}$ -in. bars at 6-in. centres, and with No. 4 $\frac{1}{8}$ -in. horizontal transverse binding bars, and No. 2 $\frac{1}{4}$ -in. bars at decking level to form a beam at the top of slab.

From the waling level 15-in. by 15-in. uprights are formed to carry the decking, at a level of 12'50 O.D. This decking is 9 in. thick and is reinforced with $\frac{1}{4}$ -in. bars at $6\frac{1}{2}$ -in. centres, with short continuity bars on the upper surface at 13-in. centres, over the cross beams, which are reinforced with No. 4 $1\frac{1}{8}$ -in. bars, of which two are bent up to take the shear. The straight bars are each fitted with No. 8 "Keedon" stirrups of $\frac{1}{8}$ -in. metal.

Arrangements have had to be made for concreting around two 3-ft. storm-water and sewer outfalls without impeding the flow in any way.

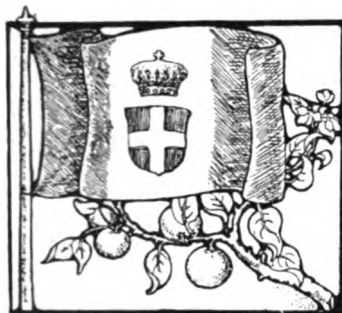
Spiral bond bars are used throughout for the reinforcement, and these being of special steel, stresses of 16,000 lbs. per sq. in. are allowed; $\frac{3}{8}$ -in. wire links are used in all beams and piles as binding.

Some delay in the construction has been caused by the rough weather, especially the gale on December 28th last, when a considerable amount of shuttering and casing was carried away; and on account of the double tides in Southampton Water, work on the waling and the driving of piles can only be carried on for an hour or two at each low tide.

The construction is being carried out under the supervision of Mr. A. V. Dibble, A.M.Inst.C.E., with Mr. J. Godden as Clerk of Works, Mr. W. Watterson representing the contractor.



Fig. 6. Completed Bays.
REINFORCED CONCRETE QUAY WALL, SOUTHAMPTON.



REINFORCED CONCRETE IN EARTHQUAKE · DISTRICTS.

REGULATIONS BY THE MINISTRY
OF PUBLIC WORKS, ROME.

Technical and Hygienical Rules for the repairing,
rebuilding, and new buildings of public and private
edifices in certain earthquake districts in Italy.

Following upon the earthquake of December, 1908, and also from experience gained in previous earthquakes, the Ministry of Public Works in Rome issued in 1909 some special rules for buildings in earthquake districts. These rules are comprehensive and very strict and cover, of course, all forms of construction, but in this article we only reproduce an abstract from the document, which abstract deals with reinforced concrete in the form of some calculations based on an example of a two-story structure in reinforced concrete. We also reproduce in abbreviated form a few of the introductory remarks and general rules.—ED.

Introduction.

THE disastrous consequences of the last earthquake are certainly to be attributed in great part to the unsatisfactory building conditions in the localities affected. The narrow streets, the excessive height of the buildings and their bad construction, the inferior quality of the materials used, were, in fact, so many causes that prepared the way for, or at least considerably aggravated, ruin and disaster.

It must also be admitted that the gravity of the disaster was not so much due to a lack of rules as to the non-observance of the rules in force.

In view of these facts, the Government in January, 1909, decided that suitable *compulsory* rules should be issued, based on the experience of past disasters; these rules to be applied to buildings under repair, in course of rebuilding, or entirely new public and private structures.

For the time being these rules are only compulsory in the communes mentioned in the Decree; all the other communes of Calabria and the province of Messina affected by the earthquake and not mentioned must await the conclusions of the Commission charged to ascertain whether reconstruction can be permitted in these districts, and, if so, under what special restrictions.

Generally.

The rules approved by the Royal Decree of April, 1909, are divided under six heads: (1) New Buildings; (2) Re-building; (3) Repairs; (4) Observance of Rules already in Force; (5) and (6) General.

As far as new buildings are concerned, the following are some of the main general principles to be applied:—

Art. 1 prohibits the construction of buildings within the limits of soil of varying nature or on unstable or boggy soil, or on ground having a steep decline. In the following article the maximum height of the buildings and the number of floors are determined.

In view of the fact, however, that for various reasons—as, for example, public utility, industrial or artistic reasons—a greater height than 10 m.

might be necessary for buildings not intended for habitation, *Art. 3* allows for this eventuality, providing always that special care is exercised in the erection of such higher buildings.

Such buildings must in all cases be completely isolated and must not exceed 16 metres in height, unless the special purpose for which they are designed—as, for example, in the case of lighthouses—absolutely necessitates a greater height.

The new rules permit of any type of construction, provided that the limits of height are adhered to and the building is provided with a complete framework standing by itself from foundation to roof and made solid by horizontal bearing structures.

A construction of simple masonry is permitted for buildings of one floor only, on condition that special precautions are used.

Whatever system of framework is adopted, however, it is indispensable that the calculations be made with great care, in order that the various parts may offer the required resistance.

[*Note*.—The rules contain various examples with calculations to serve as guidance.]

Special attention is directed to the foundations, and *Art. 4* prescribes that new buildings must stand as far as possible on naturally solid ground or ground that has been made suitable by special means.

Art. 5 prohibits the use of fragile materials or materials that are not suited for resisting special strains and compression to which they may be subjected in seismic regions, and for this reason sack masonry and masonry of natural flint stones are excluded, unless the latter are suitably broken and used under special conditions.

Arts. 10 to 17 deal with the various parts of which a building is composed, and they exclude the use of arched vaults above ground level, floors, ceiling coverings, joining together of the main framework of the buildings, the filling in and the final covering of the skeleton of frame buildings or one-story buildings are carefully and accurately worked out.

The width of streets is also dealt with, so as to provide that, should buildings collapse entirely, there will always be a space in the centre of the road free from débris. It was considered that a width of 3.50 m. would be sufficient for such clear space.

Reinforced-Concrete Structures.

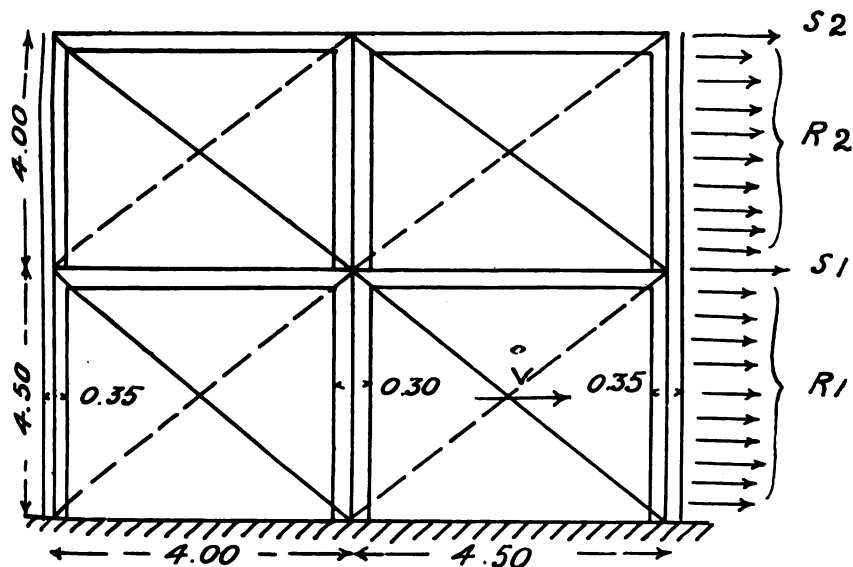
In dealing with reinforced concrete, an example is taken of a two-storied building with a double row of rooms built of reinforced concrete with a projecting rigid framework.

The principal dimensions of the transverse sections of the building (indicated in another part of the rules) are here assumed to be as follows :—

Height of ground floor ... $h_1 = 4.50$ m.

Height of the first floor ... $h_2 = 4.00$ m.

Width of the chamber, measured from the extreme ornamentation of the boundary walls to the middle of the intermediary wall, and based on this the



following calculations are given. The framework consists of three rows of pilasters. Those corresponding to the two rows of the façade have a squared section of 35 cm. per side, while the pilasters of the intermediary row are 30 cm.

We will suppose that the pilasters of one and the same row are distant 4 m. from axis to axis. The length of the compartment, which has a complete transverse frame composed of three vertical supports, the beams of the floor, and the covering and the stiffening walls will therefore be the same.

These walls may be of a single or doubly reinforced concrete, with a space between to be filled up, in which case the insertion of the indispensable number of diagonals is understood.

The ground floor is composed of a floor in reinforced concrete of the ordinary type, the first floor of a terrace also in reinforced concrete, which may be made lighter by the use of hollow bricks with a protecting layer of asphalt over them.

It is supposed that the floor and the terrace and the walls (no matter what their structure) weigh on an average 250 kg. per sq. m.

The value of 2,500 kg./m³ is adopted as the weight for reinforced concrete.

Analysis of the Weights of a Chamber of the Building 4 m. long.—Weight of the walls of the ground floor: 2 pilasters, $2 \times 0.35 \times 0.35 \times 4.50 \times 2,500 = 2,750$ kg. 1 pilaster, $0.30 \times 0.30 \times 4.50 \times 2,500 = 1,010$ kg.

Weight of the walls on the ground floor obtained by multiplying the combined development of the length on the surface of the walls themselves by the height of 4.50 by the weight per sq. m. of 250 kg. :—

$$(8.50 + 12 - 4 \times 0.35 - 2 \times 0.30) \times 4.50 \times 250 = 20,810$$

Therefore the total weight of the walls of the ground floor :—

$$2,750 + 1,010 + 20,810 = 24,570 \text{ kg.}$$

The weight of the floor or of the flat roof :—

$$8.50 \times 4 \times 250 = 8,500 \text{ kg.}$$

Total weight of the walls of the first floor, the height of which is equal to $\frac{4}{8.50} = \frac{8}{9}$ that of the walls of the ground floor :—

$$\frac{8}{9} 24,570 = 21,840 \text{ kg.}$$

Calculation of the horizontal forces for the entire Chamber in Tons :—

$$R_1 = \frac{1}{12} 24.57 = 2.05 \text{ t.}$$

$$S_1 = \frac{1}{12} 8.50 = 0.71 \text{ t.}$$

$$R_2 = \frac{1}{8} 21.84 = 2.73 \text{ t.}$$

$$S_2 = \frac{1}{8} 8.50 = 1.06 \text{ t.}$$

Calculation of the Bending Moment.—At the floor level, first floor :—

$$M_2 = (S_2 + \frac{1}{2} R_2) h_2 = (1.06 + \frac{2.73}{2}) 4 = 9.68 \text{ t.m.}$$

At floor level, ground floor :—

$$M_1 = R_1 \frac{h_1}{2} = 2.05 \times 2.25 = 4.612$$

$$+ S_1 h_1 = 0.71 \times 4.50 = 3.195$$

$$+ R_2 (h_1 \frac{h_2}{2}) = 2.73 \times 6.50 = 17.745$$

$$+ S_1 (h_1 + h_2) = 1.06 \times 8.50 = 9.010$$

$$M. = 34.562 \text{ t.m.}$$

Stresses in Vertical Supports due to foregoing strains.—In consequence of the bending moments calculated above, the corner vertical supports are subjected the one to compression stress and the other to tension stress by a strain which is respectively of the value of $\frac{9.68}{8.50 - 0.35} = 1.19 \text{ t.}$ at the first floor and $\frac{34.56}{8.50 - 0.35} = 4.24 \text{ t.}$ at the ground floor.

To the stress just calculated must be added that due to the weight of the part of the building under consideration, the weight of the walls of the ground floor only being excluded, this being transmitted directly to the foundations.

The following loads bearing on the cross section of the pilasters of the façade at floor level of the ground floor are deduced from the analysis of the weights :—

Pilaster to ground floor (the weight of the two pilasters being

$$2.75) \frac{2.75}{2} \dots \dots \dots = \text{t. } 1.38$$

$$\text{Pilaster to the first floor, } \frac{8}{9} 1.38 \dots \dots \dots = \text{t. } 1.22$$

Weight of each wall of the façade of the first floor (4-0.35)

$$4 \times 0.25 \dots \dots \dots = \text{t. } 3.65$$

Pressure exercised by the transverse walls, floor, and flat roof, always admitting the zones of concentration to be limited by the middle line of the sides, $\frac{1}{4} [2 \times 8.50 + (8.50 - 2 \times 0.35 - 0.30) 4 \times 0.25]$ $\dots \dots \dots = \text{t. } 6.12$

$$\text{Total } \dots \dots \dots = \text{t. } 12.37$$

Increasing the above-mentioned pressure by 50 per cent., in order to allow

for the dynamic effect of the heaving shock, and adding to it the stress due to the horizontal actions, the maximum compression is obtained :—

$$1.5 \times 12.37 + 4.24 = 22.79 \text{ t.}$$

Reducing this by 20 per cent. to allow for the decrease of weight that may be produced by the vertical acceleration, and subtracting from it the strain due to the horizontal actions, we have the minimum compression :—

$$0.8 \times 12.37 - 4.24 = 5.66 \text{ t.}$$

Effect of the Local Actions.—Bending produced by the horizontal forces distributed along the pilasters of the façade.

Horizontal stress applied to each pilaster of the façade at the ground floor corresponding to the mass of the pilaster itself, to that of the front wall and to half the transverse wall, supposing that its action is subdivided equally between the three vertical supports :—

$$\frac{1}{3} [1.38 + (4 - 0.35) 4.50 \times 0.25 + \frac{1}{3} (8.50 - 2 \times 0.35 - 0.30) 4.50 \times 0.25] = 0.69 \text{ t.}$$

The bending moment produced by the above-mentioned stress is considered, according to custom for the calculation of the half-morticed joists of reinforced concrete, equal to

$$\frac{1}{10} 0.69 \times 4.50 = 0.310 \text{ t.m.}$$

Calculation of the Pilasters.—Limiting the enquiry to the pilaster of the ground floor façade, we have :—

Maximum compression	...	...	...	= t.	22.79
Breaking stress $\frac{1}{3} (R_1 + S_1 + R_2 + S_2)$	...	...	...	= t.	2.20
Bending moment	...	...	...	= t.m.	0.310

Considering the very advantageous hypothesis, however, on the basis of which the enquiries have been conducted, and taking into account the inevitable deficiencies of the connections due to the openings in the walls, it is not wise to make too great an allowance by this method, which is an exceptionally favourable method of considering the resistance of a building.

The pilasters of the reinforced concrete are therefore calculated as if they were not reinforced, and with a safe stress of 25 kg./cm.². The result of this, a being the side of the squared section of each pilaster, is

$$\frac{22,790}{a^2} + \frac{31,000 + 6}{a^3} = 25$$

and therefore $a = 34 \text{ cm.}$

Provision must be made for the disposition and percentage of the framework by the usual criteria, the solidarity of the various resistant parts of the framework being better ensured, however, than usual.

Calculation of the Diagonal Framework.—If the stiffening were obtained for the whole or in part by means of diagonal frameworks, they would be calculated on the basis of the breaking stress, corresponding to the horizontal section which it meets at half length.

At the ground floor we have breaking stresses at 2.25 m. from the floor :—

$$V = \frac{1}{2} R_1 + S_1 + R_2 + S_2 = 5.52 \text{ t.}$$

If, therefore, there is a single diagonal in each quadrilateral network, or, better, if, there being two, it is supposed that only the compressed diagonal

works, we have as an expression of the strain, the inclination of 45° being preserved :—

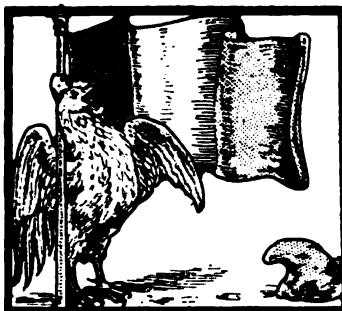
$$\frac{1}{2} 5 \cdot 22 \sqrt{2} = 3 \cdot 91 \text{ t.}$$

Having recourse to the formulæ of the solid uppermost weights for the maximum length of the diagonal $l = 450 \sqrt{2}$ cm., and the intermediary hypothesis between that of the morticed extremities and that of the said extremities being free to turn being considered as acceptable, we have for a square section of side a (everything being expressed in cm.) for a coefficient of security equal to $\frac{1}{8}$:—

$$3910 = \frac{1}{8} 2 \pi^2 \frac{E}{l^2} \frac{1}{2} a^4$$

A squared section of about 14 cm. a side is therefore deduced, to which a more or less strong framework will be applied, according to whether the diagonal is intended to resist tension also, and whether the wall does or does not give a certain confidence.

Calculation of the Floor and Terrace Beams.—This does not present any singularity in respect to the usual calculations of buildings in armoured concrete, except in regard to the appreciation of stress weight, which must be assumed to be 50 per cent. greater than its effective value.



THE WAR IN RELATION TO BUILDING.

[Contributed.]

The great War has had such terrible consequences for so many towns and villages in the areas affected by military operations that it is to be feared that the future operations of the armies will have equally serious results in many towns still untouched. The war on two fronts, for which Germany had so cleverly prepared, has already resulted in the loss of hundreds of thousands of lives, the wholesale destruction of houses, buildings and towns, and a fabulous loss of material wealth generally. What may happen in the future it is impossible to do more than surmise. The future damage, however, to structures and towns in the legitimate conduct of the war is likely to be enormous; and, as the matter is of considerable interest to constructional engineers and concrete specialists from several points of view, the following observations by a correspondent during a recent visit to a part of the war-devastated area may be of interest, as affording some idea of the effect of bombardment and fire on the towns affected.—ED.

ONE of the first impressions on entering a bombarded or destroyed town is of desolation. What was once a busy town, with conveniently planned buildings and well-arranged streets, is now a tumbled-down collection of bricks and mortar, twisted ironwork, bent girders, and calcined stonework, giving the suggestion of earthquake and fire and subsequent demolition on a large scale.



Fig. 1. Senlis after the Passage of the Germans, September, 1914.

THE WAR IN RELATION TO BUILDING.

One of the places reached and partly destroyed by the Germans in September last is Senlis, a charming little city situated on the Nonette, about one hour north from Paris by train. Here an opportunity is afforded of seeing what has been done to peaceful towns and of observing the effect of bombard-

ment and incendiarism, though, as already stated, fortunately only one part of the town has been destroyed, the old and most interesting portions escaping with little or no injury. The railway station has been entirely gutted by fire,



Fig. 2. The Rue de la République.



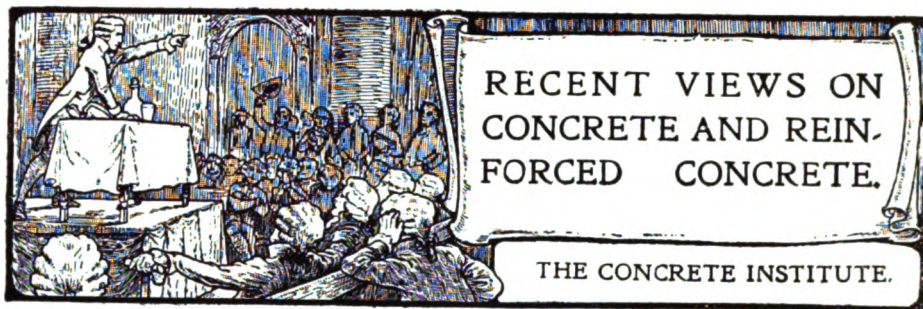
Fig. 3. The Faubourg St. Martin.
THE WAR IN RELATION TO BUILDING.

though the main walls are practically intact, and outside the building for some distance there is nothing to remind one of war. It is not until the principal street of the town, the Rue de la République, is reached that the work of

the invader is seen at its best, or worst. Smashed and burnt buildings are seen on both sides of the entire street, with an occasional structure, more or less untouched, left standing. Ruin and desolation confront one at each step: roofless dwellings, with boarded-up windows and doors, charred beams, twisted ironwork, heaps of brick and stone—desolation complete. Some buildings, of course, have withstood the ordeal better than others, and some stand to illustrate what may be termed the “curiosities of resistance,” but the destruction is so thorough, and is such emphatic evidence of a determination to destroy, that it is surprising that the main part of the town is untouched. The cathedral has sustained some damage, but that can be restored in a short time at a trifling cost, while the rebuilding of the Rue de la République, the Rue Bellon, and the Faubourg St. Martin will take months unless an army of workmen is employed. When it is remembered that what has been done in Senlis has been carried out with very much more thoroughness and much more completeness in hundreds of other towns and villages in France and Belgium, it will be realised that there will be full scope for the arts of peace when the arts of war are at rest. The demands that will be made for skilled workmen and for building materials of all kinds may have the effect of increasing cost, but it will without doubt keep engineering and building industries busy for a long time to come, and will necessitate the erection as an immediate necessity of buildings to serve simple requirements at as low a cost as possible and which shall be fire-resisting as well. A visit to towns in the war area affords ample evidence in support of pre-conceived ideas, especially that nothing can withstand bombardment with modern shells, provided that the bombardment is properly carried out, but that reinforced-concrete work does not collapse in the hopeless way in which ordinary buildings do. Moreover, with buildings subjected only to fire, reinforced concrete has an immense advantage over other materials.



Fig. 4. The Sous-Prefecture, Rue de la République.
THE WAR IN RELATION TO BUILDING.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

THE CONCRETE INSTITUTE.

SOME NOTES ON WIND PRESSURE.

By R. GRAHAM KEEVILL, A.M.I.Mech.E., Works Department, Admiralty.

The following is an abstract from a Paper read at the Fifty-eighth Ordinary General Meeting of the Concrete Institute.

WIND, as we all know, is caused by the horizontal, vertical, and rotatory movements of the atmosphere which are set up by the alterations of density, temperature, vapour, and possibly gravity.

The atmosphere as far as present-day knowledge carries us extends upwards from the surface of the earth, gradually becoming rarefied until it merges into the eternal interstellar space. Luckily, we are only concerned with movements of the air comparatively near the earth, otherwise our subject would be vast indeed.

It may be said that our modern knowledge extends backwards only so far as the destruction of the original Tay Bridge in 1879.

It will be interesting to recall, briefly, the disastrous Tay undertaking as far as it concerns us, and see what the engineers of those days—35 years ago—knew about wind pressure.

The bridge was opened for traffic on May 30th, 1878, the total cost being about £350,000. It consisted of lattice girders, with one exception of large span, carried on groups of cast-iron columns braced together, which in turn were carried on masonry foundations. The rail level at the highest point was 88 ft. above high-water level, and there were altogether 85 spans.

On December 28th, 1879, whilst a train was on the bridge and a "gale of exceptional severity was blowing," one or more of the piers gave way, bringing down eleven 245 ft. span and 227 ft. span girders, precipitating the train and 74 people into the Tay. A Board of Trade Inquiry was held, and it was stated in the Commissioners' conclusions that "engineers in France made an allowance of 55 lb. per sq. ft. for wind pressure, and in the United States an allowance of 50 lb."

The evidence given showed that the knowledge of wind and wind pressure at this period was in a state of chaos.

An endeavour was at once made by a Board of Trade Committee to put matters on

a satisfactory footing. After examining wind records, they on May 20th, 1881, recommended :—

“That for railway bridges or viaducts a maximum wind pressure of 56 lb. per sq. ft. should be assumed for the purpose of calculation.”

This immediately came into force for railway work, and was also adopted generally for structures on public works, although such adoption was purely arbitrary, or specified in local by-laws.

Much opposition was raised to this standard of pressure, because it was considered excessive and not justified by experience of existing structures. As far as buildings are concerned, the horizontal pressure assumed for designing has gradually been reduced to the 30 lb. per sq. ft. now often taken.

There are three methods for the measurement of wind :—

- (i) Velocity measured by the Robinson anemometer.
- (ii) Pressure measured by a pressure tube anemometer.
- (iii) Pressure measured by a plate or board set face to the wind.

Considering (i), the anemometer that bears his name was invented by Dr. Robinson in 1846, and is by far the most used of the various instruments which have been produced from time to time. It consists of four arms at right angles; at the end of each arm is a hemispherical cup. In the standard pattern the cups are 9 in. diameter, whilst the arms are 2 ft. long. The arms are mounted on a spindle, which rotates with the movement of the cups in a wind, and is geared to mechanism for counting the revolutions made. The cups are set to face all one way, so that the concave face of one cup is always presented to the wind, and consequently as the wind pressure within the cups is greater than on their convex backs, rotation is caused by quite a small wind. This instrument therefore measures direct velocity, and Dr. Robinson, after experimenting, stated that the velocity of the cups was one-third the velocity of wind for all instruments whatever their size. This is now known to be a fallacy, with the result that practically all records of velocity prior to 1902 were incorrect and generally much too high.

The discovery that the factor, 3, for the Robinson anemometer was incorrect is due to the experiments carried out by Mr. Dines, and since confirmed by other authorities.

These experiments consisted of placing anemometers on horizontal arms, which were rotated by means of a steam engine; $14\frac{1}{2}$ revolutions of this “whirler” constituted a half a mile of space, whilst great care was taken to ensure accuracy in the results. The number of turns of the whirler and the revolutions of the anemometers, both of which were counted automatically, determined the relation between the distance travelled and the revolutions of the anemometers, and thus the value of the new factor, 2·2, was obtained.

The outcome was that the Wind Force Committee of the Royal Meteorological Society in 1902 adopted the factor 2·2 instead of 3 for the Standard Robinson anemometer. Thus when the old factor was used, and the instrument indicated, say, a velocity of 60 miles per hour, the use of the new factor reduced the velocity to 44 miles per hour. Converting these velocities into pressures by the formula $P = 0\cdot003 V^2$, the 60 miles per hour velocity gives 10·8 lb. per sq. ft., whilst the velocity of 44 miles per hour gives a pressure of 5·8 lb. per sq. ft. This rotating anemometer has one great disadvantage, in that owing to the inertia of the instrument it does not record gusts of wind, which may have a much greater velocity than the average values shown by this instrument.

Another type of instrument is the pressure tube anemometer invented by Mr. Dines. This instrument automatically records all the varying pressures of the wind, and consequently all gusts and lulls that occur during a gale whatever the velocity of the wind may be. The principles upon which the design of this instrument is based are that when the wind blows into the open horizontal mouth of a tube it causes an increase of pressure within it and in all air-tight arrangements connected with it. On the contrary, if the wind blows horizontally across the open mouth of the vertical tube,

the column of air within the tube rises and the pressure is reduced within it, and in all air-tight arrangements connected with it. The great advantage of this instrument is that it not only gives a record of all phases of wind, but a practically perfect exposure can be obtained by mounting it on a high pole.

The third type of anemometer is like the second type, a pressure recorder, and is in general use, although now practically obsolete. This type consists of a board or plate which by means of a weather vane is kept facing the wind. The back of the board is fitted with springs which keep the board up to its work. The wind acting on the face of the board drives it backwards, compressing the springs; the pressure is then measured and recorded automatically by the movement of the board or the compression of the springs.

This type of pressure boards was used at the Forth Bridge during its construction.

When Sir B. Baker, in conjunction with Sir John Fowler, undertook the responsibility for this great structure, it became necessary to ascertain as nearly as possible what would be the probable wind pressure on the bridge in the exposed position on which it is sited.

He accordingly erected, on the top of the Old Castle on Inchgarvie, three anemometers. The principal one was 20 ft. long by 15 ft. high, approximately the size of the side of a railway carriage of those days, erected vertically with its surfaces facing east and west, this direction being the most unfavourable from which the wind can strike the bridge. This board was hung in a frame and was carefully adjusted and balanced and fitted with four springs at the four corners to adjust the board to its normal position. The movement of the board was communicated to the registering apparatus by means of wires.

To ascertain to some extent the effect of small local gusts plates were fitted of 18 in. diameter in holes cut out of the centre of the board and at the top right-hand corner, and arrangements made to register the pressure on them separately.

A circular plate with an area of $1\frac{1}{2}$ sq. ft. was erected to face the same direction, and 8 ft. away from the large board, and provided with means for registering the pressure.

The third was a revolving gauge also with an area of $1\frac{1}{2}$ sq. ft. and fitted with a vane so that it always faced the wind.

The results from the Forth Bridge are interesting and valuable in that they show records of an exposed position at considerable differences in height and width. From a consideration of them with other reliable information now available, they appear to be high in value and to be influenced by gusts and possibly by some momentum of the plates of the anemometers.

In order to see at a glance the results valuable to us, the velocities have been reduced to pressures by the formula $P = 0.003 V^2$.

It will be of interest to note that the greatest velocity recorded since the Meteorological Office has been at South Kensington is 54 miles per hour, on July 28th, 1911, which is equivalent to a pressure of about 8.75 lb. per sq. ft.

The relation between coast and inland winds is a practical question which often arises when an important structure is to be designed in some locality where building regulations are of an elementary nature, or non-existent, and where the only guide is the possession of reliable information, but which has been taken at one or more exposed stations near the coast, whilst the proposed structure may be sited some distance inland. Dr. Shaw says that "speaking from the daily experience of weather maps we may say that a westerly wind of force 8 at an exposed western station will be represented at inland or east coast stations by a wind force of 5. The corresponding reduction of velocity is from 42 to 21 miles per hour. Provisionally, therefore, we may say that the surface wind velocity (that is winds that will affect a structure) at a reasonably well-exposed inland station, or at a coast station for an off-shore wind, is reduced to one-half the velocity of an open coast station." As pressure is dependent on velocity it follows that the pressure will be reduced to a corresponding amount.

When dealing with a building of large exposed area there is often a difference of opinion, in the absence of regulations, as to what wind pressure to adopt—i.e., whether the maximum average velocity or the greatest gust. There is little available information

as to what may be termed the area of a gust, but it is considered to be comparatively small. In a well-designed and well-constructed structure it would appear to be good enough to take the maximum velocity, and trust to the continuity of the structure to distribute local gusts. In Professor Henry Adams's engineers' handbook there is a table for reduction of wind pressures according to heights and widths of a building.

The knowledge of wind at low and high altitudes—the latter concerns the flying men more than anybody else—has made great progress in recent years. Investigations on a large scale have been carried out by Dr. Shaw's department, and it is now possible to calculate the velocity of the wind at high altitudes, when the surface velocity is known. "As a rule (but with exceptions) the wind increases rapidly as the surface is left, up to two, three, or four times the surface velocity. The increase of wind appears to be in direct ratio with increase of height.

"For the station at Ditcham Park it has been found that—

$$V_h = \frac{h}{h_0} V_s$$

where V_h = velocity at height h above sea-level,
 h_0 = height of a well-exposed anemometer above sea-level,
 V_s = velocity recorded by the anemometer.

"The corresponding law for other places is not yet known. There is, however, a consensus of opinion that velocity increases with height, by factor and not by constant addition. A likely formula is:—

$$V = \frac{H+a}{a} V_s$$

where H = a height above ground,
 V = velocity at H ,
 V_s = velocity recorded by the anemometer,
 a = constant.

"From the examination of observations at various stations the following curious conclusion has been drawn: the constant a for the purposes of rough approximation may be taken to be *the height of the station above sea-level.*"

Another method of obtaining a knowledge of the wind is by a system of observation.

In 1805 Admiral Beaufort, Hydrographer of the Navy, devised the scale of wind force which bears his name.

With the passing of the sailing ship and the present universal use of steam the Beaufort scale has lost entirely its original significance. The scale, instead of passing into oblivion together with the ships that it was designed for, has become rejuvenated, and is now the British standard for estimating wind force on land and sea all over the world.

We are not concerned with wind at sea; the land part of the specification is as given in the accompanying table.

In the report of the Meteorological Committee for the year 1906 it is stated that during the investigation into the Beaufort scale it was found that when "the velocities arrived at empirically, as the appropriate equivalents of the Beaufort numbers, were plotted upon a diagram and a working curve drawn to represent them, the curve so drawn turns out to be very nearly identical with that represented by the algebraical equation $V = 1.87 \sqrt{B}$, where B is the Beaufort number and V is the hourly wind velocity in statute miles per hour. Adopting Mr. Dines's factor 0.003 for obtaining the wind pressure in pounds upon a square foot, it follows that the relation between the Beaufort numbers and the wind pressure, P , is expressed by the comparatively simple relation—

$$P = 0.0105 B^3.$$

Beaufort Number.	Description of Wind.	Specification of Beaufort's Scale for Use on Land.	Mean Wind Force in lb. per sq. ft. at Standard Density. $P = 0.0105 B^3$.	Velocity in miles per hour. $V = 1.87 \sqrt{B^3}$.	Probable Maximum Velocity attained by Wind in miles per hour.
1	2	3	4	5	6
0	—	Calm; smoke rises vertically	0	Averages.	1.5
1	Light breeze	Direction of wind shown by smoke drift, but not by wind vanes	0.1	2	4
2		Wind felt on face; leaves rustle; ordinary vane moved by wind	0.8	5	9.5
3		Leaves and small twigs in constant motion; wind extends light flag	0.28	10	15
4	Moderate breeze	Raises dust and loose paper; small branches are moved	0.67	15	24
5		Small trees in leaf begin to sway; wavelets form on inland waters	1.31	21	30
6		Large branches in motion; whistling heard in telegraph wires; umbrellas used with difficulty	2.3	27	28
7	Strong wind	Whole trees in motion; inconvenience felt when walking against wind; umbrellas discarded in exposed places	3.6	35	46.5
8		Breaks twigs off trees; generally impedes progress	5.4	42	56
9		Slight structural damage occurs (chimney-pots and slates removed)	7.7	50	66
10	Storm forces	Seldom experienced inland; trees uprooted; considerable structural damage occurs	10.5	59	78
11		Very rarely experienced; accompanied by widespread damage	14	68	—
12	Hurricane	—	Above 17	Above 75	—

"Thus, if all the steps in this intricate investigation have been accurately followed, there appears to be underlying Admiral Beaufort's original specification the simple relation that the numbers selected are proportional to the cube root of the wind pressure."

The question of the relation of wind velocity to wind pressure can now be considered.

The formula already quoted in this paper is $P = KV^2$, where P equals the pressure per sq. ft., V the velocity of the wind in miles per hour, and K a constant.

It is the value to be taken for K that has caused a vast amount of discussion and research in recent years.

It can now be accepted that for flat solid surfaces placed at right angles to the wind the value for K is 0.003 or 0.0032.

The value for K obtained by Dr. Stanton is the result of an elaborate series of experiments carried out at the National Physical Laboratory, and fully described by him in the *M.P.I.C.E.*, vols. clvi. and clxxi.

The conclusions arrived at were that "there appeared to be good reasons for supposing that the mean intensity of pressure on similar surfaces of area greater than 1 sq. ft., exposed to the wind, is independent of their actual dimensions."

Interesting experiments to obtain the distribution of air pressure were made by Dr. Stanton in a uniform current of air on small models representing closed buildings with roofs at 30°, 45°, and 60° to the horizontal. These may be briefly summarised as follows:—

Model with 30° roof.—Plus pressure on the windward vertical wall and over about the middle third of the slope of the roof on the windward side.

Minus pressure over the remainder of the windward roof slope, and on the leeward slope and wall.

Models with 45° and 60° roofs.—Plus pressure on the windward vertical wall and slope, except at the ridge, where it becomes minus.

Minus pressure on the roof and wall of the leeward side.

Whilst these results were obtained with an artificial current of air, Dr. Stanton says that "the action of wind is no doubt more complicated than that of a uniform current of air; yet it seems probable that in displacements of considerable intensity, such as gales, the conditions of a uniform current may be approximately fulfilled, so that the distribution of the wind pressure on exposed structures may be regarded as that due to a uniform current."

Further experiments were carried out on a model roof with slopes of 56 sq. ft. in area, and so arranged that the angle of the slopes could be varied.

Values for K for use in the formula $P = KV^2$, where P equals the normal pressure on the roof in lbs. per sq. ft., and V equals the velocity of the wind in miles per hour, were obtained, so that it is only necessary to know the maximum velocity of the wind at any site in order to arrive at the pressures on a proposed building at the site.

Another phase of the investigations at the National Physical Laboratory was an endeavour to throw light on the much-vexed question of the shielding effect of members of a lattice girder placed behind each other, and the effect of a windward girder on a leeward girder of a bridge.

From the small models it was found that with two circular plates placed $1\frac{1}{2}$ times their diameter apart the total pressure was "less than 75 per cent. of the resistance of a single plate."

On increasing the distance to "approximately 2·15 diameters the total pressure was equal to that on a single plate."

Again increasing the distance to "5 diameters, the total pressure was only 1·78 times that on a single plate."

"Experiments on square and rectangular plates gave corresponding results, it being found that the shielding effect of long rectangles was considerably less than in the case of circular plates, but in all cases the maximum shielding effect was observed when the plates were at a distance apart of approximately 1·5 times the least cross dimension."

Investigations were made on model lattice girders subjected to the uniform current of air. The "experiments were made on a single girder, and on two parallel girders with and without a roadway, normal and inclined to the direction of the current. The shielding effect of the windward girder is very considerable, for both normal and inclined currents. When the two girders are connected by a roadway and are at a distance apart equal to the depth of the girder the pressure on the leeward girder is 15 per cent. of that on the windward girder, and at twice this distance the pressure is 25 per cent. of that on the windward girder. The effect of the roadway appears to diminish the total pressure on the girders. When the direction of the current is inclined to the plane of the girders the resultant normal pressure is increased for small angles of obliquity, but not to any great extent, the value of the normal pressure when the current makes an angle of 75° with the plane of the girder being approximately 5 per cent. greater than that for normal incidence."

The regulations relating to wind pressure on bridges have already been mentioned. In the L.C.C. (General Powers) Act, 1909, we find that: "For a roof the plane of which inclines upwards at a greater angle than twenty degrees with the horizontal the superimposed load (which shall for this purpose be deemed to include wind pressure) shall be estimated at twenty-eight pounds per square foot of sloping surface." Also, "All buildings shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than thirty pounds per square foot of the upper two-thirds of the surface of such building exposed to wind pressure."

Turning to the proposed L.C.C. Regulations for Reinforced Concrete—which may some day come into force—it is stated, "For a roof the plane of which inclines upwards at a greater angle than twenty degrees with the horizontal the superimposed load, which shall for this purpose be deemed to include wind pressure and weight of snow and ice, shall be estimated at twenty-eight pounds per square foot of sloping surface on either side of such roof."

"All buildings shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than twenty pounds per square foot of the whole projected surface normal to the direction of the wind.

"All structures or attachments whatsoever in connection with a building, including towers or other parts which extend above the roof flat or gutter adjoining thereto, shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than 40 lb. per sq. ft. of the whole projected surface normal to the direction of the wind."

It is impossible to include the regulations of the foreign nations and at the same time keep this paper within respectable limits, but those for the chief towns of the United States are of especial interest, because of the great height of some of the buildings.

The principal wind regulations for the United States are summarised in the following table:—

Town.	Horizontal Wind Pressure in lbs. per sq. ft.		Allowable Increase of Stress in Wind Bracing and Members Subjected to Wind Loads.	Remarks.
	Exposed Position	Sheltered Position		
	Per cent.	Per cent.	Per cent.	
New York	30	30	50	{ Except those under 100 ft. in height in which the height does not exceed four times the average width of base.
Rochester				
Minneapolis				
Louisville				
Atlanta				
Newark	30	25 to 35	30 25	{ 25 lb. at tenth story less 2½ lb. for each story lower and 2½ lb. additional for each story above to a maximum of 35 lb.
Jersey City				
Philadelphia				
Syracuse				
Detroit	30	30	25	—
Baltimore	30	30	none	—
Buffalo				
Chicago	20	20	50	—
San Francisco				
St. Louis	30	30	20	—
Seattle	None specified		50	—
Boston	Provision for wind bracing sh all be made where necessary.			
Providence	No regulations			
Los Angeles				

The amount of published matter on this subject is considerable. An endeavour has been made to embody within this paper the reliable information that is of practical use.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

DESCRIPTION OF REINFORCED CONCRETE GALLERY AT THE EMPIRE THEATRE, WHITBY.

This gallery, although not the largest constructed of its kind, is of particular interest on account of the long cantilevers employed to carry same.

Upon reference to the plan, *Fig. 2*, it will be seen that the width of the gallery is 39 ft. and its mean depth about 33 ft. Between the front of the gallery and the front wall of the building there is a 9-in. brick wall.

Reference to the cross section, *Fig. 3*, will show how the position of this wall was taken advantage of to dispense with main beams by constructing two reinforced concrete columns in the thickness of the wall to carry the two cantilevers shown in *Fig. 2*. These cantilevers have a length of 18 ft. 6 in. from centre of column to face of curb, and give a good idea of the possibilities of this combination of materials for constructional work.

The two front cantilevers are balanced by two cantilevers at the back of the gallery, which are anchored at their ends into the mass concrete footings of the front wall, in order to prevent any rotating tendency of the front cantilevers when loaded first.

The strip footing to the columns is 2 ft. wide by 18 ft. long. The beam to this footing is 3 ft. deep by 9 in. wide with toes on either side 6 in. thick. The columns are 9 in. by 16 in., stiffened by a 9 in. by 9 in. tie at the level of the underside of cantilevers.

The cantilevers at the back are 4 ft. 6 in. deep by 12 in. wide, and those at the



Fig. 1. View of Finished Building.
THE EMPIRE THEATRE, WHITBY.

front taper from 4 ft. 6 in. deep at the columns to about 10 in. at the front, and are likewise 12 in. wide throughout. There is a curb round the front 6 in. by 5 in., and over this the balustrade wall itself 3 in. thick. The stepping to the gallery has a tread of 30 in. and a rise of $7\frac{1}{2}$ in.

At the back of the gallery there is a distributing beam 18 in. by 9 in., and each back cantilever has a member 8 in. by 8 in., which forms a cover to the anchor bars referred to before.

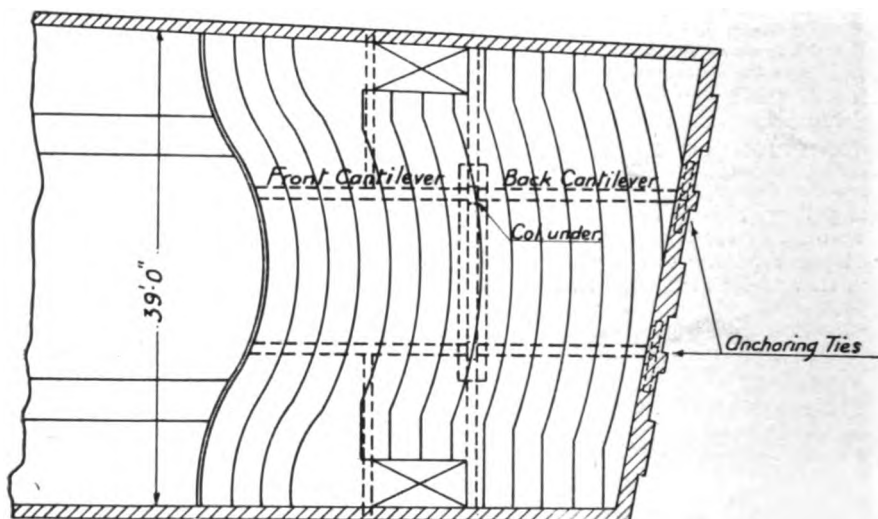


Fig. 2. Plan.

REINFORCED CONCRETE GALLERY, EMPIRE THEATRE, WHITBY.

Fig. 4 is a photograph of the completed gallery, and Fig. 5 a photograph of the underside of same, which clearly shows the 18 ft. 6 in. long cantilevers.

The staircase leading to the gallery was also constructed in reinforced concrete.

This structure was designed to carry an evenly distributed load of 1 cwt. per superficial foot (in addition to its own weight) with a factor of safety of four, and proved to be perfectly satisfactory when tested.

The designs were prepared by the Indented Bar and Concrete Engineering Co., Ltd., of Westminster. The reinforcement was entirely of indented bars.

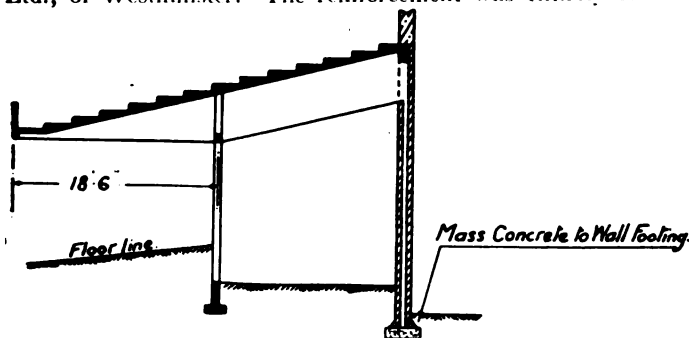


Fig. 3. Section.

REINFORCED CONCRETE GALLERY, EMPIRE THEATRE, WHITBY.

The architect for the building was Mr. A. E. Young, 77, Baxtergate, Whitby. The work was carried out by Mr. John Cooke, of Huddersfield, as contractor.

CONCRETE COTTAGES AT WROTHAM.

THE Wrotham Tenants, Ltd. (Chairman, Sir Mark Collett), have had some

cottages built at Battlefield, Wrotham, Kent, by the Concrete Block Company, Ramsgate, Billericay, Essex.

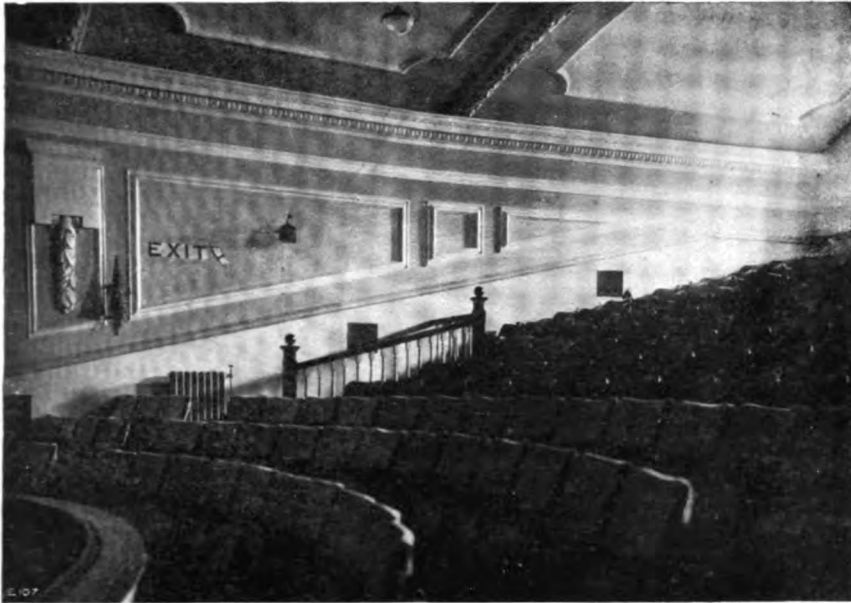


Fig. 4. View of Completed Gallery.



Fig. 5. View showing Underside of Gallery and the 18 ft. 6 in. Cantilevers.
REINFORCED CONCRETE GALLERY, EMPIRE THEATRE, WHITBY.

The estate is situated about one and a-half miles from Wrotham Station.

Fourteen cottages in all (seven pairs) have so far been erected, and it is stated that they have proved very satisfactory; the erection of further cottages is, therefore, under consideration.



CONCRETE COTTAGES, WROTHAM, KENT.

One pair of cottages was designed by Mr. Arnold Mitchell, and the cost of same is £138 each. The accommodation comprises two living rooms, 9 ft. 3 in. by 8 ft. 3 in., and 14 ft. 8 in. by 9 ft. 3 in.; a scullery 8 ft. by 8 ft.; and there are three bedrooms measuring 11 ft. 3 in. by 8 ft., 7 ft. by 8 ft. 9 in., and 15 ft. by 8 ft. 3 in. respectively.

Two pairs were designed by Messrs. Stanley Barrett and Driver. These have

worked out at £156 per cottage, and the ground floor of each contains two living rooms, 9 ft. 9 in. by 9 ft. and 12 ft. 9 in. by 12 ft.; further, a scullery, 8 ft. 6 in. by 8 ft., and on the first floor there are three bedrooms measuring 15 ft. 9 in. by 9 ft., 9 ft. by 8 ft., and 12 ft. by 7 ft.

Two pairs were designed by Messrs. McDermot and Co., and cost £183 each cottage. The accommodation here comprises two living rooms, 11 ft. by 10 ft. and 11 ft. 9 in. by 13 ft. 11 in. respectively; further, a scullery, 5 ft. 6 in. by 7 ft., and three bedrooms of the following dimensions: 16 ft. 9 in. by 9 ft., 10 ft. 2 in. by 8 ft., and 13 ft. by 8 ft. 6 in.

The remaining two pairs were designed by Mr. Pinkerton and cost £196 each cottage. These contain a living room, 10 ft. 8 in. by 10 ft. 4½ in.; a kitchen, 12 ft. 6 in. by 13 ft. 6 in.; a passage, 5 ft. 9 in. by 10 ft. 8 in.; a scullery, 4 ft. 9 in. by 12 ft.; and three bedrooms, 10 ft. 8 in. by 13 ft., 11 ft. 3 in. by 8 ft., and 9 ft. 6 in. by 10 ft.

All the cottages were erected entirely of concrete. The outside walls were built of 9-in. hollow concrete blocks made on an American block machine and on a "Winget" machine supplied by Messrs. Winget, Ltd., of Newcastle-on-Tyne. The inner walls were built of concrete slabs made partly on a "Winget" machine and partly on a "Perfection" machine, the latter being supplied by the Star Concrete Machinery Co., 8, Manor Park, Lewisham, S.E.

The blocks and slabs were made on the site and consisted of three parts breeze, two parts sand, and one part cement. The outside walls were cement and lime rough cast.

The ground floors were finished with 1-in. floor boards nailed on direct to the pitched and tarred breeze concrete.

All the cottages were roofed with red "Record" concrete tiles, also made on the site on a "Record" machine supplied by the Star Concrete Machinery Co. It is stated that the low cost of the buildings was to some extent due to the use of these tiles, which are made of two or three parts clean sharp sand and one part cement. We are informed that they have proved waterproof and stormproof.



CONCRETE COTTAGES, WROTHAM, KENT.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

British Standard Specification for Portland Cement. (Report No. 12, Revised March, 1915.)

Published for the Engineering Standards Committee, 28 Victoria Street, Westminster, S.W., by Messrs. Crosby Lockwood & Son, 7 Stationers' Hall Court, Ludgate Hill, E.C. Price 5s. net

A new edition of the British Standard Specification for Portland Cement, one of the most widely adopted of the Reports issued by the Engineering Standards Committee, has just appeared. The Sectional Committee on Cement, presided over by Sir William Matthews, K.C.M.G., was formed in 1903, and the first Report was approved by the Main Engineering Standards Committee and issued in 1904. The specification has been revised from time to time with a view of improvement in the quality of the cement, and the new edition just published contains further important modifications.

Increased fineness of grinding of cement has been legislated for—the maximum residue permitted on the 180² mesh and 76² mesh being reduced to 14 per cent. and 1 per cent. respectively—and the minimum tensile strength at 7 days of both neat cement and cement and sand has been raised to 450 and 200 lbs. per square in. respectively. The clauses dealing with the preparation of the briquettes have been amplified, the procedure being described in greater detail than has previously been the case. In the case of both neat cement briquettes and cement and sand briquettes ramming or hammering is expressly prohibited, but the wording of these clauses still leaves scope for wide differences of opinion as to the quantity of water to be used and the precise methods of filling the moulds. The fact that the cement is only required to be plastic "when filled into the moulds" should enable gaugers to avoid the low and irregular results which await those using too great a quantity of water.

The growth of tensile strength required at 28 days both for neat cement and cement and sand is now obtained by means of a formula in place of the fixed percentages which have been specified hitherto. The formula gives a progressive increase in-

stead of the somewhat irregular steps of the method previously adopted, but has the effect of requiring a somewhat increased percentage of growth on the higher 7-day figures. In the case of the neat tests 30,000 would probably have been a more equitable factor than 40,000.

With the view of making easier the determination of what constitutes a visible impression in the test for final setting time the Vicat needle for this test is provided with a fixed metal circular cutting edge set half a millimetre back from the point, as this length is sufficient to clear the scum which sometimes forms on the surface of the pat. The cement is considered to be finally set when the needle makes an impression on the pat and the attachment fails to do so. The attachment is illustrated on the plate showing the Vicat needle.

All the plates have been entirely redrawn, and two new plates have been added to the specification, one dealing with a standard spatula for use in making up the cement and sand briquettes, and the other shows an improved form of specific gravity bottle which the Committee have approved as suitable for use, though they do not stipulate for its employment to the exclusion of any other pattern. Instructions for obtaining the specific gravity of cement are given on the plate.

Spon's Architects' and Builders' Pocket Price Book.

Publishers: E. F. N. Spon, Ltd., 57 Paternoster, S.W. Price 2s. 6d. net.

This price book is issued in a convenient size for pocket use. It is most valuable as a reference book to architects, surveyors, and others interested in the building trade or its various branches. It has been brought up to date, and contains a great amount of new matter.

Extensive revisions have been made in the prices, which are in accordance with the cost of material at the time of going to press.

The trades are in the order adopted in a well-drawn bill of quantities. Any information required is easily found from the fully detailed index to the trades.

Concrete Stone Manufacture. By Harvey Whipple.

Detroit. Concrete-Cement Age Publishing Co. 255 pp.
Price \$1.00.

Contents.—The Development of Concrete Building Units — Location, Equipment, Layout—Materials, Mixtures, Manipulation — Curing — Special Moulds and Patterns—Surfaces—Shop Records and Cost Keeping—Building Regulations — Tests—Specifications—Selling the Products—Examples of Layout and Operation.

The author of this volume deals with factory-made concrete units, which, as he states, have not developed as rapidly as field-made concrete in mass; although factory conditions are more easily controlled than field conditions. There is no doubt that there is some possibility of concrete unit construction becoming more general in the future, but this industry has been seriously hampered in the past by many who have taken up the business without sufficient capital or enterprise and have been satisfied to put on the market inferior articles of limited application which have been no recommendation to the building owner. If this type of construction is to make any real headway it must be possible to obtain first-class products capable of extensive application by numerous standard types which do not impose too many restrictions on the user. Mr. Whipple fully appreciates these facts, and he has endeavoured to help the industry by the suggestions and advice contained in this book. In dealing with the question of location, he gives the two important considerations which must be taken into account, viz., (1) The market for the product; (2) the availability of suitable raw materials; and he follows this section with details of the equipment and methods necessary to produce good material. Many drawings and photographs are given which enable the reader thoroughly to understand every point. The materials employed are then described, and this is followed by an important chapter on curing, which contains much valuable information, and we cannot speak too highly of this section. In the chapter devoted to surface treatment the author is very frank in his

statements as to the lack of beauty in concrete blocks and units, but at the same time he calls upon manufacturers to eliminate this disadvantage in the future, and supports his call with many practical suggestions as to the attainment of the object, and the illustrations given of work actually executed, where the appearance has been fully considered, certainly indicate great possibilities. After dealing with questions of obtaining good work of all classes, the fact that the manufacturer must make profits and build up a lucrative business is not overlooked, and this aspect is considered on practical lines by the author, who displays a thorough knowledge of the whole subject. The book is well written, and is a comprehensive treatise on concrete block construction which can be thoroughly recommended.

The Ternary System $\text{CaO}-\text{Al}_2\text{O}_3-\text{SiO}_2$. By G. A. Rankin. With Optical Study by Fred E. Wright.

The American Journal of Science. 79 pp.

Contents.—Introduction—Review of the Binary Systems—Phase Rule and its Application to Ternary Systems—Experimental Methods—Experimental Investigation—Recapitulation: The Concentration—Temperature Model—Some General Considerations—Application of the Equilibrium Diagram to Problems Involving the Three Oxides—Summary.

This is an extremely scientific little book, which will necessitate some scientific knowledge on the part of the reader if he is to make an intelligent study of the matter contained therein. It is the first complete attempt to determine all the compounds, both binary and ternary, made up only of CaO , Al_2O_3 , SiO_2 , and the mutual relations of these compounds; and the investigations, of which this book is the record, necessitated some 7,000 heat treatments and subsequent optical examinations of the products. The data obtained are made use of in a discussion of the nature and constitution of Portland cement clinker, but, although the matter is well expressed and the diagrams are good, the book does not appear likely to be of much service to engineers generally.

INDUSTRIAL NOTES.

These pages have been reserved for the presentation of articles and notes on proprietary materials or systems of construction put forward by firms interested in their application. With the advent of methods of construction requiring considerable skill in design and supervision, many firms nowadays command the services of specialists whose views merit most careful attention. In these columns such views will often be presented in favour of different specialities. They must be read as ex parte statements—with which this journal is in no way associated, either for or against—but we would commend them to our readers as arguments by parties who are as a rule thoroughly conversant with the particular industry with which they are associated.—ED.

STEEL PILING.

In recent years the Dock and Harbour Boards have made considerable use of steel piling in carrying out extensions or new works.

Among the more recent examples of the employment of steel piling is the work at the West Hartlepool Docks, carried out by the North Eastern Railway, under the Chief Engineer for Docks to that Company, Mr. Chas. Watson, of Hull. The scheme comprises the modernisation of the North Basin with its entrances from the Old Harbour and the Central Dock.

The old walls, with curved faces, will be replaced by vertical walls in concrete faced with blue brick and Cornish granite, the depth of water will be increased, and the entrances widened from 60 ft. to 70 ft.

To effect this it was necessary to construct two large external cofferdams, one about 420 ft. linear at the Old Harbour entrance, the other about 230 ft. linear at the Central Dock entrance, as shown in Figs. 1 and 2, and the following conditions had to be met, involving much forethought and skill on the part of the North Eastern Railway Dock engineers:—

The depth of water in the Outer Harbour is 33 to 35 ft. at high tide, with over 16 ft. rise and fall, and north-east gales are prevalent in winter. Further, to secure sufficient depth for excavating the foundations of the new dock entrances, it was

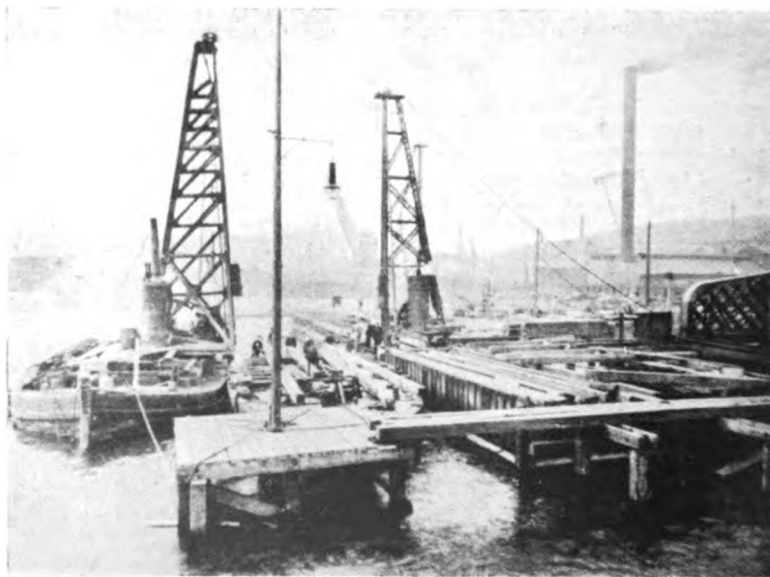


Fig. 1. SHOWING STEEL PILING COFFERDAM AT OLD HARBOUR ENTRANCE.

necessary to penetrate very deeply, in some cases as much as 30 ft., into hard boulder clay.

These conditions involved the use of piling in 50-ft. to 60-ft. lengths, and capable of withstanding exceptionally heavy driving.

After careful enquiries and inspection, the steel piling supplied by the Side Groove Steel Piling Supply Co., Ltd. (Annison's Patent) was selected. It has a strong interlock, wide centres, and an independent pile bar which can be carried down as a king pile below the sheeting members. (See Fig. 3.)

The transport of these 60-ft. lengths presented slight difficulties, easily overcome, however, by the use of bolster wagons.



FIG. 2. STEEL PILING COFFERDAM AT CENTRAL DOCK ENTRANCE.

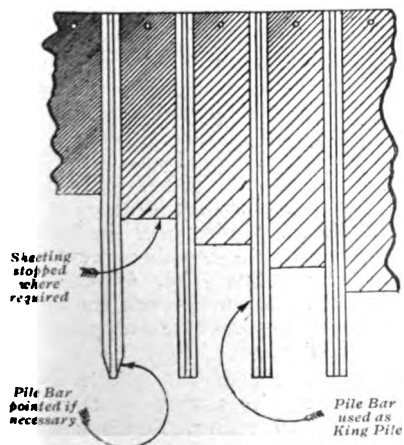


Fig. 3.

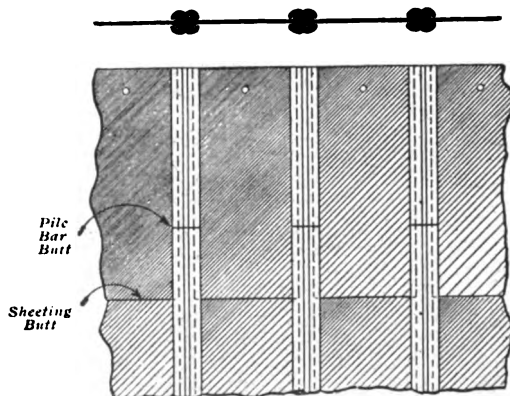


Fig. 4.



Fig. 5. 60 FT. WHOLE-LENGTH BENT CORNER PIECES.



Fig. 6. DRIVING 60 FT. WHOLE-LENGTH ASSEMBLED PILE BAR AND SHEETING AS ONE.

In this connection, it is stated that in a few instances where deflection of the sheeting-members had occurred during transport, this was quite removed by the assembling.

For cases where such lengths cannot be transported shorter lengths are used, the lengths being carried beyond the butts of the oval bulb sheeting-members, forming a sleeve-joint of great strength, the sheeting being fish-plated. (See Fig. 4.)

In driving the two sections, a pile bar and sheeting-member being assembled by sliding the oval-bulb of the latter into one groove of the pile bar, the two were lifted together and driven as one, the free groove of the pile bar passing over the bulb of the sheeting-member previously driven thus (Fig. 6) effecting $21\frac{1}{4}$ in. linear of work each time the piling machine was placed in position.

The 60-ft. whole-length corner pieces were bent to the required radius, the assembled sections engaging and driving freely, and the penetration of the

piling into the extremely hard North East Coast Boulder Clay from 25 to as much as 32 ft. was most successful, the piling well sustaining the hard driving experienced.

Fig. 7 shows the completed Old Harbour Entrance cofferdam pumped out, and gives a good idea of its condition.

Full particulars of the steel piling here described can be obtained from the Side Groove Steel Piling Supply Co., Ltd., 16, Water Lane, Great Tower Street, London, E.C., who are shortly opening additional offices for the benefit of their many West End clients, in Victoria Street, S.W., as soon as the necessary alterations are completed.

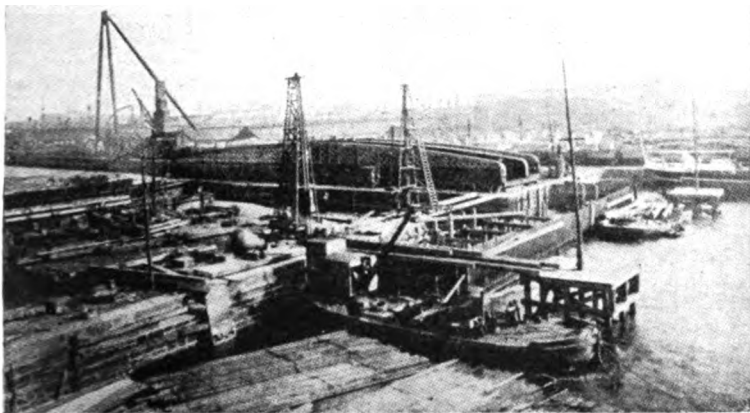
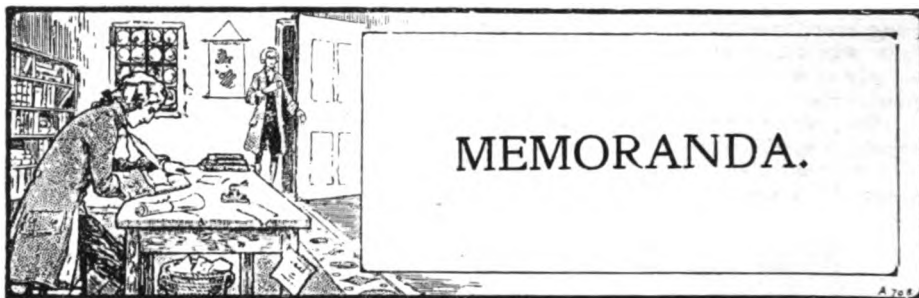


Fig. 7. SHOWS OLD HARBOUR ENTRANCE WITH COMPLETED 450-TON STEEL PILING COFFERDAM.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Iron and Steel Institute.—The annual meeting of the Institute will be held, by kind permission, at the Institution of Civil Engineers, Great George Street, Westminster, on Thursday and Friday, May 13th and 14th, 1915, commencing each day at 10.30 o'clock a.m. In the event of Mr. Greiner being unable to attend, Mr. Arthur Cooper, LL.D. (immediate Past-President), will preside at the meeting.

The council have decided that, on account of the War, it will be inadvisable to hold the annual dinner this year.

It has been provisionally decided that the autumn meeting shall be held in London during the week ending September 25th. Further particulars of the arrangements will be announced in due course.

The Institute of Arbitrators.—Our attention has been called to the above Institute, which has been formed in London at the instance of members whose services are usually invoked for the purpose of acting as arbitrators in commercial matters, in references under partnership agreements, building and other trade contracts, with a view to their corporate association both for their own benefit and in the interest of the general public.

Among the objects of the Institute are the following:—

(1) To support and protect the character, status, and interests of the profession of arbitrator generally.

(2) To afford means of communication between professional arbitrators on matters affecting their various interests.

(3) To arrange periodical meetings for the reading of papers and the discussion of matters connected with the duties and responsibilities of arbitrators, etc.

The annual subscription is £1 1s. The members of council at present are Professor Henry Adams, M.Inst.C.E., Mr. Max Clarke, F.R.I.B.A., Mr. F. Malcolm Barr, M.S.A., and Mr. I. W. Bullen, F.L.A.A. The president and vice-president have not yet been nominated.

Fuller particulars can be obtained from the secretary, Mr. H. C. Emery, at the temporary offices of the Institute, 32, Old Jewry, E.C.

Warrington Bridge.—This bridge, regarding which we gave a short description some time ago, has now been completed. It is built entirely in reinforced concrete on the Considere system, and forms an important connecting link between Lancashire and Cheshire. It has taken three years to build, and cost £23,370. A commemorative tablet was affixed to the bridge, and this tablet was unveiled last month by the Mayor of Warrington, who stated that the bridge was the second widest in Great Britain, and was only 2 ft. less than Westminster Bridge. It will be remembered that it is 80 ft. across between the parapets, and has a clear space of 134 ft.

The White City.—The Government tenancy of the White City ended in April, and the exhibition authorities have, we understand, decided to use the property for factory accommodation, which is much needed at present. The buildings can be well and easily adapted for this purpose.

New Pennsylvania Elevator of Reinforced Concrete at Philadelphia.—A large amount of American and Canadian wheat for export is hauled by the Pennsylvania

from connecting railroad lines reaching the western wheat belt and from Buffalo and Erie, where it is received from Lake steamers, to Girard Point at the mouth of the Schuylkill River near Philadelphia. This grain is there loaded directly on to ocean-going vessels or stored temporarily in the new reinforced concrete elevator which was recently put into service, replacing an old frame elevator which had been in use about thirty years.

The plant consists in general of a track shed in which the grain is unloaded; a working-house in which it is elevated, weighed, and distributed; a set of 34 circular and 40 intermediate concrete storage tanks in which it may be held temporarily; a loading gallery extending out over the pier; a drier house for the treatment of damp or heated grain, and a transformer house in which the high voltage electric current received from the Philadelphia Electric Companies' lines is transformed for use in operating the elevator. The plans contemplate the extension of the storage bins, and the duplication of the track shed and working-house alongside the present layout.

The loaded cars are received in a 12-track yard about 600 ft. long, which converges to six tracks about 800 ft. long extending over the twelve unloading hoppers in the track shed to an 11-track load yard about 500 ft. long. The cars are pushed through to the load yard south of the track shed by a locomotive, and are pulled back, as required for unloading, by a six-drum Webster car puller. After unloading, the cars are given a kick by the puller which starts them down the 2 per cent. grade to the main yard. A double set of crossovers, with double slip switches between the track shed and the yard, allows the empties from any unloading track to be sent in on any yard track.

The track shed consists of a reinforced concrete slab roof 120 ft. by 144 ft. in size, covering six unloading tracks spaced 20 ft. centre to centre, and supported at an elevation of 23 ft. 6 in. on the low side by rows of concrete columns between the tracks. Each of the six tracks passes over two unloading hoppers in the track shed into which the grain is dumped by Clark car shovel machines. The track hoppers are arranged in four transverse rows of three each, served by four belt conveyors in the basement below the hoppers. The unloading facilities are designed to handle 250 cars in ten hours.

The working-house is a concrete building 62 ft. 6 in. by 94 ft. 6 in. adjacent to the track shed. It is 202 ft. 6 in. high, and contains a basement, first floor, working bins 74 ft. 6 in. high, and a four-storey cupola. Each of the receiving conveyors dumps the grain into a steel boot tank, from which it is elevated by one of the four receiving legs to the top of the working-house at a maximum rate of 15,000 bushels per hour for each leg. All elevating legs are operated by rope drive from electric motors.

The receiving legs empty into 2,000-bushel concrete garners at the level of the top floor in the cupola, which deliver the grain to 2,000-bushel hoppers on 60-ton standard scales. After weighing, the grain may be distributed to the storage bins, to the drier house or the separators, to a car for re-shipment, or to the working-house bins for loading on boats.

Five belt conveyors on the bin floor carry grain from the working-house out over the storage tanks. If the distributing spout under any particular scale hopper will not reach the conveyor serving the bin which it is desired to fill, a reversible transfer belt just below the distributing floor may be used to transfer the grain to the proper conveyor. Self-propelled trippers dump the grain from the conveyor belts into the desired tanks.

Construction of Buildings and Tanks.—The Girard Point plant was made both fire-resisting and permanent by the use of concrete throughout. The few necessary windows have metal sash and wire glass, and the gutters and cornice are of galvanised iron. Great care was taken to secure a stable foundation, as the load to be carried when the tanks are full is very great. The adopted design, involving a solid 4-ft. concrete slab on piles under the entire structure is conservative, but was considered justifiable under the circumstances. The surface soil at the site is black mud to a depth of about 50 ft., the underlying strata being sand and, still lower, gravel. More than 5,000 yellow pine piles 60 ft. to 70 ft. long were driven on 2-ft. 3-in. centres under the working-house and storage bins and on 5-ft. centres under the track shed. These piles were designed to carry 16½ tons each, although tests showed that a load of 54 tons would not cause a perceptible settlement.

The piles were cut off at the elevation of the bottom of the slab and earth was packed around their heads. After laying the lower 9 in. of the slabs over the pile heads,

The BRITISH STEEL PILING CO



Widening Dove Bridge, Uttoxeter.

The contractor who used the piling wrote :—"I have been well satisfied with the piling."

The Piling used was our "SIMPLEX," in 18 ft. lengths, 22 lbs. per sq. ft. Sufficient was supplied to do half the work : the piles were driven through the timber foundation grillage of old work with a No. 3 McKiernan-Terry Hammer, and withdrawn with one of our "GRIPS" : then we repurchased the materials from the contractor.

Weights of Piling on sale :—

"Simplex" ...	...	22 to 27 lbs. per sq. ft.
"Universal" ...	...	from 43 lbs. " "

DOCKHOUSE, Billiter Street, LONDON, E.C.

a six-ply felt and pitch waterproofing membrane was laid, on which the remainder of the 4-ft. slab was placed. This waterproofing membrane was carried up the outside of the walls well above high-water level and was protected by a single course brick wall laid in cement. The slab under the working-house was reinforced by $\frac{3}{4}$ -in. bars laid parallel with both edges and also diagonally, crossing under the column footings.

The intermediate columns supporting the bins and upper floors of the working-house are 4 ft. in diameter and spiral reinforcement. The diagonal girder system of supporting the working bins, which was developed by Messrs. James Stewart and Co. in 1906, and has been adopted for several of the new elevators, was used.

The storage tanks are 15 ft. in inside diameter and 96 ft. high with 7-in. walls. They are supported on rectangular concrete piers resting on the continuous slab. The walls are reinforced with steel hoops and vertical rods. The total capacity of these storage tanks, including interspaces, is as follows :—

27 circular tanks at 12,900 bushels	348,300 bushels.
27 circular tanks at 13,200 bushels	356,400 "
24 interspaces at 3,400 bushels... ..	81,600 "
16 interspaces at 3,200 bushels... ..	51,200 "
Total	837,500 "

The tanks are covered by a concrete slab forming the floor of the cupola in which the conveyors from the working-house are located. The roof is a $5\frac{1}{2}$ -in. concrete slab covered by a four-ply tar and felt coating.—*The Railway Gazette.*

Concrete Surfaces.—When concrete surfaces are discoloured as a result of smoke, soot, and similar impurities, it is sometimes possible to remove the accumulation by applying naphtha. If this fails a light application of sand blast will entirely remove the discoloured surface.—*Concrete-Cement Age.*

Acid Towers Built of Reinforced Concrete.—An acid tower group of reinforced concrete has just been completed for a large paper company at Erie, Pennsylvania. Four towers, 10 ft. by 99 ft., set in a square, 15 ft. centre to centre, form the group, which is made a unit by a common foundation slab and a housing at the top embracing all four towers. The towers are lined with special tile laid in acid-proof mortar to resist attack from the acidulated liquor dripping through the stone filling of the towers. The footing slab is 32 ft. square by 6 ft. thick, its horizontal reinforcement is engaged by the vertical rods of the tower reinforcement anchoring the towers. The stack shells are 9 in. thick with $\frac{1}{4}$ -in. vertical and $\frac{5}{8}$ -in. rings of square twisted rods. The lowest tower section has 20 vertical bars in the circumference, but at the top these are decreased to 12 bars. The rings are spaced 5 in. apart in the lower section and 20 in. apart in the top.

The height is 99 ft. from foundation to tower top. A floor over the towers carries a housing 25 ft. 4 in. square by 7 ft. 6 in. high where rock is stored. Over the housing in turn is a water tank 11 ft. square by 3 ft. high. Walls and columns support the roof.—*Engineering News.*

PROPOSED NEW WORKS.

New Mill.—The Urban District Council propose constructing a reservoir of 1,500,000 gallons capacity, in regard to which the Local Government Board are holding an enquiry.

Windsor.—A scheme has been sanctioned by the Corporation for carrying six filters on a reinforced concrete raft resting on concrete piles.

Selsey.—Tenders have been invited by the Selsey Water Co. for the construction of a water tower or reservoir of 50,000 gallons capacity upon piers forty feet above the ground level, to be constructed on the "Kahn" system of reinforced concrete.

Doncaster.—The Doncaster Rural District Council are considering tenders for the extension of their existing reservoir at Barmborough from 40,000 to 120,000 gallons. The work is to be executed in reinforced concrete to the competing firm's designs.

Midlothian.—A concrete storage tank and other works for the supply of water to the Gorebridge district is to be constructed by the Gala Water District Committee of the Midlothian County Council.

Haddington.—Tenders have recently been lodged with the Town Council of Haddington for a concrete storage tank of 450,000 gallons capacity.

Pontypridd.—The Ystradyfodwg and Pontypridd Main Sewerage Board propose laying a cast iron pipe sewer on concrete piers in the River Rhondda at Hopkinstown.

ACCEPTED TENDERS.

The following tenders have recently been accepted :—

Bury.—Extension to reinforced concrete wall at the Electricity Generating Station for the Corporation : Bolton Stone and Concrete Co.

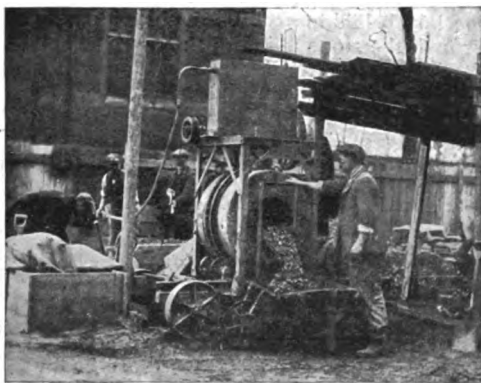
Keith.—Construction of Cuthil reservoir for the Town Council : W. Cruickshank Broomhill Road, Keith.

Hull.—Construction and maintenance of reinforced concrete service tank and water tower at Holderness for the Hull Corporation : T. Goates and Son, £3,173.

CONTRACTS OPEN.

London.—Tenders are invited by H.M. Office of Works, to be in by May 10th, for the foundations and concrete retaining wall in connection with the new General Post Office, East. Specifications, etc., of the Secretary, H.M. Office of Works, Storey's Gate, London, S.W. Deposit, £1 1s.

Whitby.—The Royal National Life-boat Institution invite tenders for a timber-framed life-boat house and reinforced concrete substructure, slipway and approach at Whitby Harbour. Quantities, etc., of Messrs. Douglass, Lewis and Douglass, engineers, 15, Victoria Street, Westminster, S.W. Deposit, £1.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

The **VICTORIA** is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.
(For Description see p. 273)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X. No. 6.

LONDON, JUNE, 1915.

EDITORIAL NOTES.

THE CONCRETE INSTITUTE.

The Annual Meeting.

THE most interesting feature in connection with the Concrete Institute's Annual Meeting was the speech made by Mr. Cyril Cocking in acknowledging the Institute's medal, which was presented him for his undoubtedly excellent paper on Calculations for Steel Frame Buildings, of which an abstract was given in this journal some time ago.

The points raised by Mr. Cocking in his speech are of the greatest importance and value, and in our opinion the speech should be given prominence and publicity, and is therefore reproduced below. Mr. Cocking said:—

There were two important aspects which to his mind required the urgent consideration of the Concrete Institute. The first was the question of education.

TECHNICAL EDUCATION AND THE WAR.

At the present time the cream of the manhood of our Empire was taking a very active part in the greatest conflict of nations that the world had seen, a war that was being waged with scientific instruments which reached the highest level of human invention and human investigation. They were fighting a nation which could boast the finest educational system in the world, and the question for them, as an engineering institution and an educational institution, was, should they be ready, when the war was over and the period of recuperation and reconstruction was upon them, to take and hold the foremost position among the civilised nations of the earth? Should they be in a position to send out competent engineers and competent draughtsmen to repair the damage the war had caused, and should they be able to capture Germany's engineering trade in addition and without prejudice to the building and engineering work which was always on hand in this country? That was a problem of supreme importance to them, because it appeared to him that the nation that was going to make good and win and maintain the upper hand would not necessarily be the one that would be victorious in the field of battle, but the nation that took the shortest time to recover from the war's ill effects. The nations of Europe at the present time were like sick men on a bed, and it was the man that got up first that was going to remain the strongest. This to him was a very important point. It would mean a further war for commercial and professional supremacy, and efficiency in the engineering profession was almost, if not entirely, a question of education. It was a matter of national importance that the education of the structural and constructional engineer should be placed upon a sound and up-to-date basis. Classes of instruction should be so arranged as to give the necessary continuity of purpose in order that the deplorable wastage of students' time which now obtained might be avoided. It was

also necessary that the instruction given should be uniformly good, well prepared by the instructor, and easy of assimilation and retention by the student. In that connection he suggested that steps be immediately taken by the Institute to inquire as to the ways and means whereby the educational system of the country, particularly with regard to structural engineering, could be improved and extended in order to ensure that the British structural engineer should hold the supremacy in every quarter of the globe. There should also be means provided so that the best advice could be given to the student leaving the day-school to enable him to fit himself as a structural engineer in the shortest possible time. He was convinced that, if the Institute were to consider the education question as directly applied to their profession, and drew up a comprehensive and well-conceived scheme, the education authorities would come up to the scratch and see the thing through. The most important point was that it should be done at once. In a year or two they would be too late, because our enemies had the education that we lacked.

QUALIFICATION FOR THE STRUCTURAL ENGINEER.

The second question he desired to bring to their notice was the necessity of some qualification for the structural engineer. It seemed to him a curious anomaly that a structural draughtsman with a deficient theoretical training and incomplete practical experience should be, and often was, allowed to design, without supervision, structures upon which the safety or the lives of his fellow men depended, whereas in the medical profession a rigid training and a certificate of competency were required before a medico could commence to practise at all. In the past he had designed the steelwork for theatres and other places of public assembly, and with only the most meagre supervision, at a time when he could not regard himself as fully competent, and yet the lives of people had been and were still dependent upon the accuracy of his calculations, theories, and details. Surely the structural engineer should be required to show some certificate of competency before he was allowed to design without supervision.

Mr. Hills, in his Paper, had stated that the erection of buildings should only be undertaken by properly qualified individuals. On this statement he was persuaded that they were all of one mind, but were they to wait until some appalling disaster took place in their midst before the importance of proper qualification was brought home to the man in the street and the responsible engineer? The question of responsibility was often confused. Some people would argue that, because a public building must be designed to the design of the district surveyor, the district surveyor took full responsibility, or if a design was supervised by a consulting engineer, the latter took the responsibility. Personally, he could not accept that view; to his mind the responsibility rested with the designer, whether his work were supervised and checked or not. The designer who refused the responsibility of his own designs declared himself incompetent and unqualified. The question was, what should be the qualification of a structural engineer? That was for the Institute to decide. There should be a theoretical qualification and a practical qualification, but the one was useless without the other. Why should not the Concrete Institute issue certificates of competency to competent men? They would be of estimable value, both to employer and employee, and, incidentally, to the district surveyor. His own personal view was that it would have been better for

the engineering profession if the 1909 Act had never been promoted, but an Act of Parliament had been passed compelling structural engineers to register themselves and empowering the Concrete Institute, together with the other Institutions directly interested, to formulate rules to be observed, both in the design and carrying out of reinforced concrete and steel structures; that was, constituting their branch of the engineering profession on a basis similar to that of the legal profession and the Law Society. He knew it was very difficult to draw a comparison, but an Act like the 1909 Act did not encourage the professional man; it only encouraged the incompetent by giving him rules to work to of which he did not appreciate the value and importance; in fact, the whole aim, especially of the Reinforced Concrete Regulations, was to make a Building Act that they might call "fool-proof." The Act had recently improved design, but only up to a certain point; it did not allow for future improvement and economy, and it destroyed originality. Personally, he would much prefer that the engineer should be properly qualified and be allowed a free hand within reasonable limits.

THE LONDON BUILDING ACTS.

MR. HILLS undertook a very big task when he commenced to deal with the London Building Acts in the form of a paper before the Concrete Institute, and we consider that he accomplished his work well. The fourfold objects of the Act cover such a wide field that it is impossible to confine the clauses to a few simple requirements, and a certain amount of ambiguity is inseparable from an Act which is drawn up by lawyers in the necessary legal phraseology to prevent any possibility of evasion.

As time passes, so new conditions and methods arise, and, in consequence, an extension of the Act is necessary in the form of some amendment or other, and the course of evolution is such that this state of affairs will continue indefinitely, and we can only deal with the difficulties by passing new Acts at intervals of so many years which will eliminate for the time being the disadvantage of having to consider several Acts instead of one complete set of By-laws and Regulations.

The great difficulty which designers often have to face is the fact that the "letter" of the Act is rigidly administered while the "spirit" does not appear to have any value, and it is in this respect that it would be advisable to give district surveyors the power of using their discretion. Several instances could be given in support of this statement, but it is not necessary to prove what is a generally acknowledged fact. The point to be remembered is that every clause in the Act is, or should be, inserted to accomplish a definite object or prevent the possibility of a certain danger occurring, and the clauses are framed in such a manner that they will cover the worst possible condition if necessary.

There are many instances, however, where such clauses do not apply in the spirit owing to conditions which render their application unnecessary, but they must be enforced as they are in the Act. In many cases an application can be made to the London County Council, who recognise the fairness of the application and give consent, but time and money are so wasted because such consent could not be given by the district surveyor. If such discretion were given to district surveyors, we have sufficient confidence in them to know that

their powers would be wisely and justly used, and no concessions granted which would endanger their reputations.

The wording and arrangement of the Acts as at present existing certainly require a little more than average intelligence, and to the designer the Act often appears as a kind of trap in which he is never quite sure he will not get caught. He may wish to comply with the Act in every way, but every new scheme seems to arouse fresh sections, which were not considered applicable in previous work, and in one district the surveyor will read the Act in one way and in another district a different interpretation will be put upon it. Such a state of affairs could easily be remedied by making the Act more defined, without detracting from its controlling power. We do think that the tendency in the later Amendment Acts has been to state the requirements more clearly, and in the event of a new Act being passed to embody all the existing By-laws we hope that this tendency will be followed to the extent of reconsidering the whole of the wording and arrangement of the 1894 Act.

The discussion which followed the paper at the Concrete Institute was very interesting, and in many cases the opinions were to the point and indicative of the right principles, and we hope that the fact of the subject having been raised at the Institute will result in improvements in the near future, and thus benefit will accrue to the building owner, designer, and district surveyor, none of whom can express satisfaction with the Acts as at present existing.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

WE publish this month the first of a series of six interesting articles by Mr. Nathan C. Johnson on "The Microscope in the Study and Investigation of Concrete," which first appeared in the *Engineering Record* of the United States. Generally speaking, it may be said that these articles support and strengthen the opinions, long held in this country by those who have made a special study of the subject, as to the essential points to be observed in the production of strong enduring concrete. Thus, in his second article Mr. Johnson says:—

"In sea water construction especially it is probable that the defects which induce failure lie primarily not so much in the cement, providing cement of proper quality be used, as in the improper proportioning of aggregates, or in poor mixing, or in a combination of the two."

This endorsement of the view which has been so often urged in these columns is of peculiar value in this instance, as Mr. Johnson has approached and examined the subject in a direction which we believe has not been investigated thoroughly in earlier researches. Apart, however, from the grading, proportioning, and mixing of concrete materials, a wide field for discussion on the complex problem of the hydration of Portland cement is opened up by these articles. We should like to see this phase of the question pursued further in relation to the fineness of grinding of the cement. As is well known, the cement manufacturers of this country have been pioneers in the very fine grinding of their product, and one cannot but feel more than ever convinced, after reading Mr. Johnson's articles, that in voluntarily saddling themselves with increased cost of manufacture in order to improve the quality of their product they have probably rendered our engineers a service of the very highest value and importance.



EMPIRE AND INDIA HOUSES, KINGSWAY.

By A. LAKEMAN, M.S.A.

There are many features of architectural interest in the buildings here described, which are of steel-frame construction. The reinforced concrete features are the staircase and floors. — ED.

SOME excellent business premises have been erected in Kingsway from the designs of Messrs. Trehearne and Norman, architects, and they furnish a good example of modern building as regards planning, detail, and construction. The site is situated at the junction of Aldwych and Kingsway, and the scheme is divided into two blocks, that on the angle with return frontages to Kingsway and Aldwych being known as Empire House and the other, which has a frontage to Kingsway only, has been named India House. The latter has a frontage of 90 ft., and the former has a width of 47 ft. 6 in. to Kingsway, 34 ft. to Aldwych, and 60 ft. on the angle. A plan showing the arrangement of a typical floor is illustrated in *Fig. 1*, and it will be seen that the maximum of lighting has been obtained by the introduction of suitable areas, and the staircases and lifts are situated in convenient positions, whilst allowing the floor space to be divided up into offices when required without loss of room. Another point that is worthy of notice is the absence of internal columns or stanchions, which might prove to be obstructions in the economical division of the floor space, and in any case it is advantageous to have clear uninterrupted floors. There are eight floors above the ground floor and an additional floor in the corner dome, while the basement floor is 12 ft. deep. The total height from the pavement to the main roof is about 104 ft., and to the roof of the dome about 125 ft., as shown in the section illustrated in *Fig. 2*. The elevation is illustrated in *Fig. 3*, and it affords an excellent example of the architectural treatment of a building which is intended solely for commercial purposes. It is obviously far more difficult to produce a fine architectural effect in a building of this class than in a monumental building, on account of the limitations placed upon the designer, and all the more credit is, therefore, due when a successful result is achieved, as in this instance.

Owing to the omission of columns and stanchions, which would obstruct the clear floor space, the principal girders have rather long spans, and are, therefore, of a fairly heavy section. In all cases the calculations have been based on the requirements of the London County Council contained in the 1909 Act, although the work to Empire House cannot be considered as that of a steel-framed structure, the walls conforming to the 1894 Act and not being wholly carried on steelwork at each floor level; but in India House thin walls were employed and the whole structure was designed as coming within the meaning of the Act applying to steel-framed buildings. The plan illustrated in *Fig. 5* shows the lay-out of the steelwork to Empire House at the first floor level and

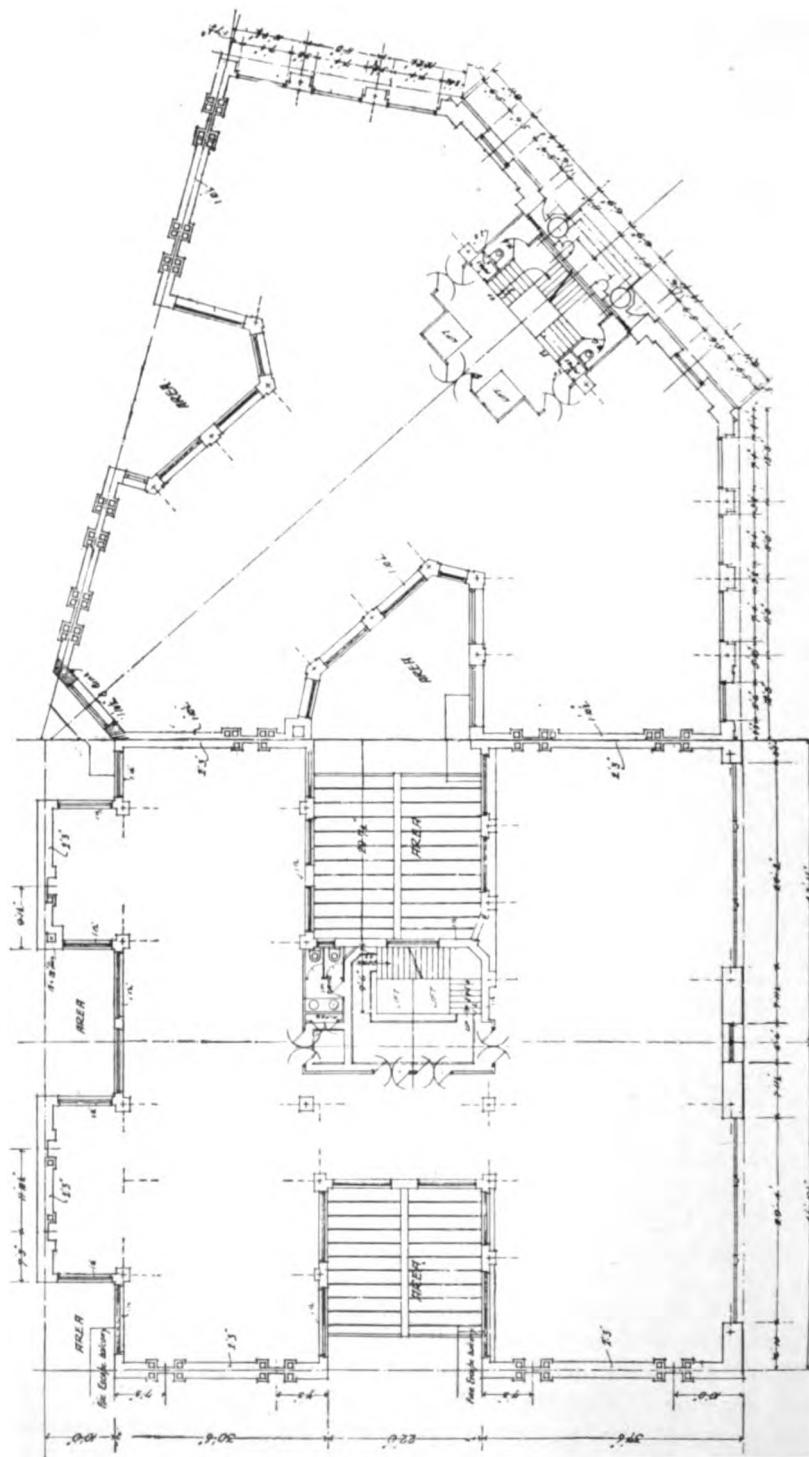


Fig. 1. Typical Floor Plan.

this can be considered as a typical detail. The steel beams are planned to divide the floors up into bays about 12 ft. wide, and the span of these beams in nearly all cases is about 34 ft., thus necessitating the use of compound sections,

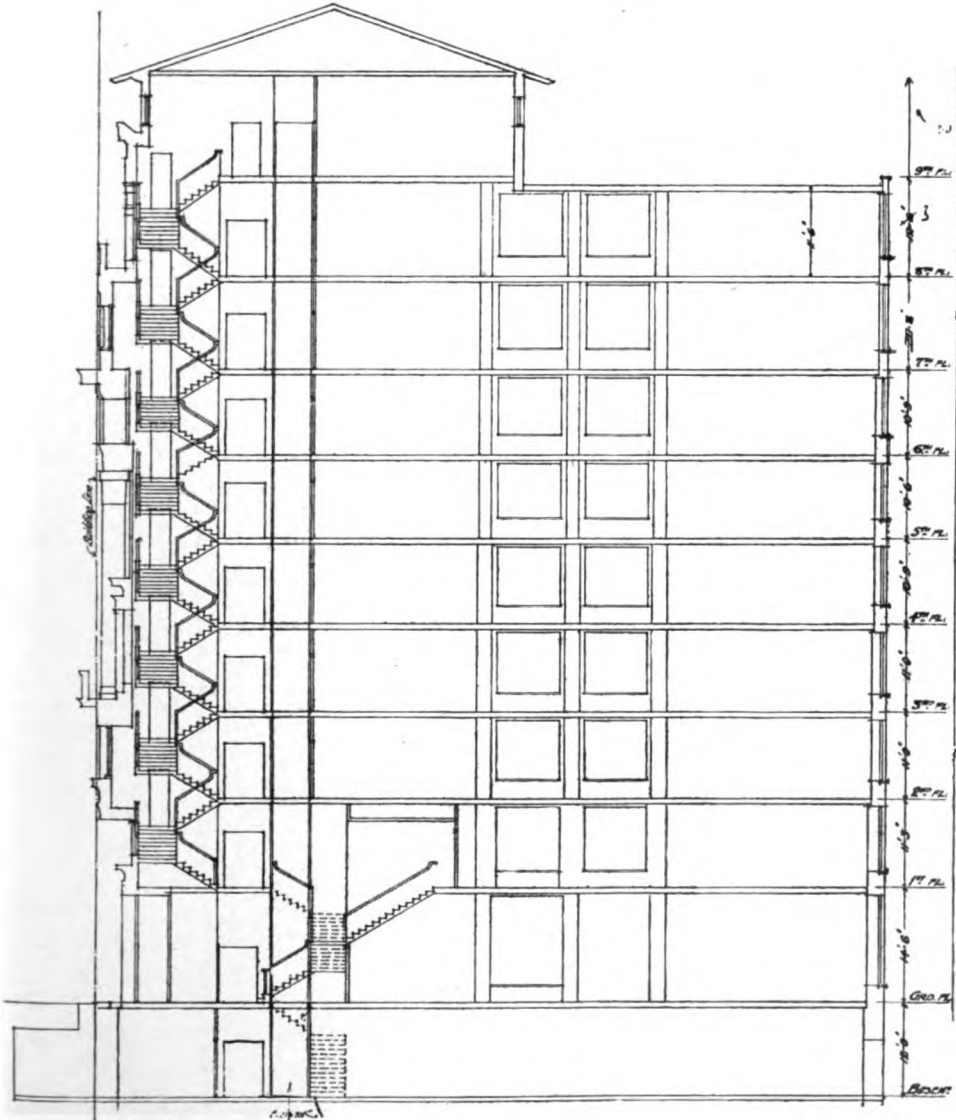


Fig. 2. Cross Section.

EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.

21 or 22 in. deep and 12 in. wide. The largest girders in the building are those carrying the main front walls at the second floor level, these having a span of about 34 ft. and supporting two concentrated loads, each about 11 ft. from the ends, such loads being caused by compound steel stanchions which occur

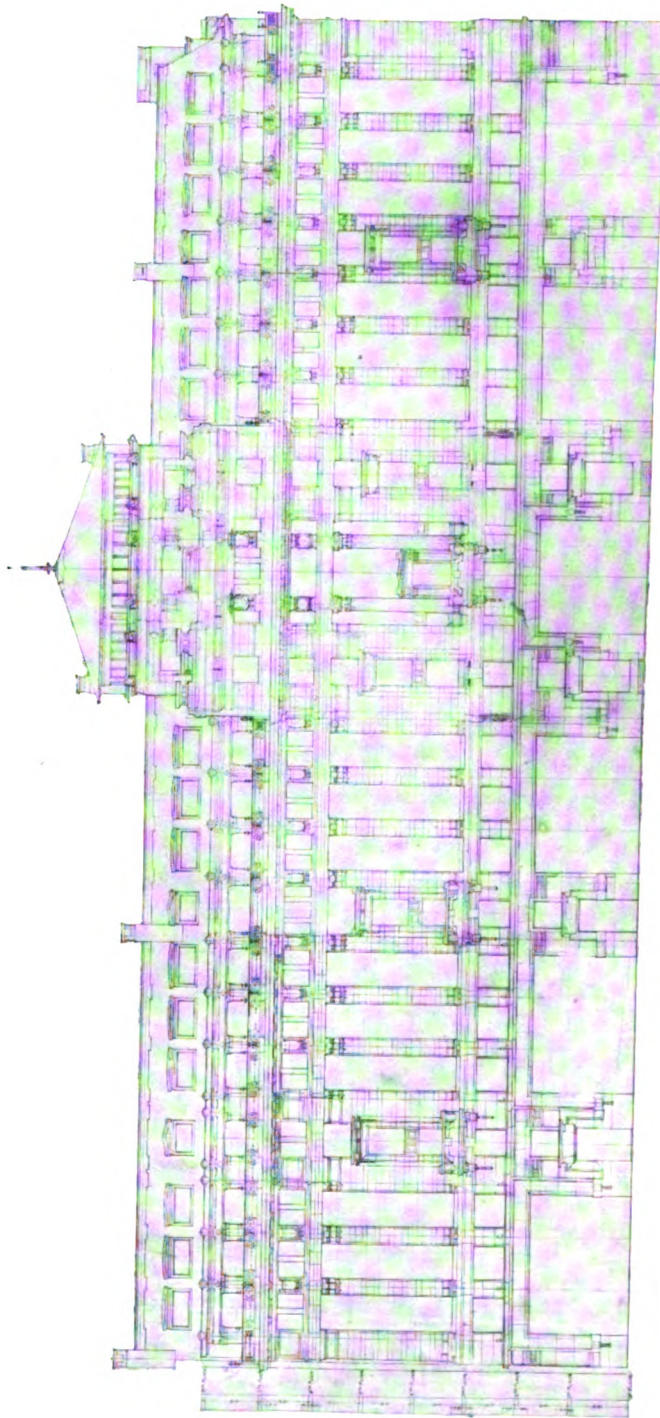


Fig. 3. Elevation of Building.
EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.

behind the piers seen in the front elevation extending through the second, third, and fourth floors. These beams are designed as box girders with a total depth at the centre of 4 ft. 6 in. and a width of 1 ft. 9 in., and they are built up with two 48 in. by $\frac{3}{8}$ in. web plates and six 21 in. by $\frac{1}{2}$ in. flange plates at top and bottom, connected to the web plates with 6 in. by 6 in. by $\frac{3}{8}$ in. angles. Angle stiffeners are provided throughout the length, and immediately under the stanchions a length of 6 in. by 5 in. rolled steel joist is placed vertically between the two web plates. A detail of this work is illustrated in *Fig. 6*. The stanchions, with typical details of connections, is shown in *Fig. 7*, and it will be seen that a grillage foundation was adopted. In the example shown three layers of steel joists were used, the top layer consisting of four 20 in. by 7 $\frac{1}{2}$ in. joists 6 ft. 6 in. long, resting on eight 12 in. by 5 in. also

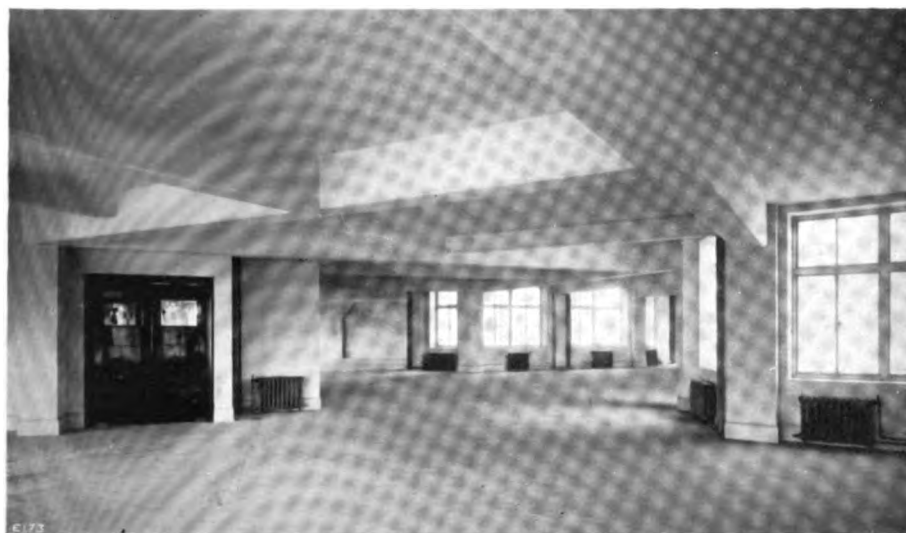


Fig. 4. Interior View.

EMPIRE AND INDIA HOUSES, KINGSWAY, 'W.C.

6 ft. 6 in. long, and these in turn are bearing on nine 6 in. by 3 in. joists which are 8 ft. long. The concrete below is 10 ft. sq. and this is carried up to the basement floor to envelop the steel entirely and prevent any corrosion occurring. The steel base to the stanchion is kept 3 ft. below the floor line and is about 3 ft. 6 in. by 2 ft. 3 in., with the usual gusset plates and angle connections. The stanchion itself is of compound section and is built up with two 12 in. by 6 in. rolled steel joists and six 14 in. by $\frac{3}{8}$ in. plates from the foundation to the first floor level, where the section is reduced by substituting four 14 in. by $\frac{3}{4}$ in. plates.

The arrangement of the steel in the octagonal dome is shown in *Fig. 8*, this being built up with 15 in. by 5 in. rolled steel joists, which act as hips, attached to a plate ring $\frac{1}{2}$ in. thick, and over these and between them a reinforced concrete filling is placed, while radiating 6 in. by 3 in. R.S.J. are suspended from the hips, and these in turn carry 5 in. by 3 in. steel

joists, from which $1\frac{1}{4}$ in. by $\frac{1}{4}$ in. steel flat bars on edge are hung with steel hangers to support the ceiling, which is executed with expanded metal and plaster. All the radiating 6 in. by 3 in. R.S.J.'s are connected to a $\frac{1}{2}$ in. centre plate, and four 2 in. by $\frac{1}{4}$ in. steel flat suspenders are provided to hold them up, these suspenders being connected to the plate ring above mentioned.

The floors throughout are constructed with reinforced concrete slabs between the main steel beams above mentioned, these having spans of about 12 ft. and being executed by the general contractor with ordinary round

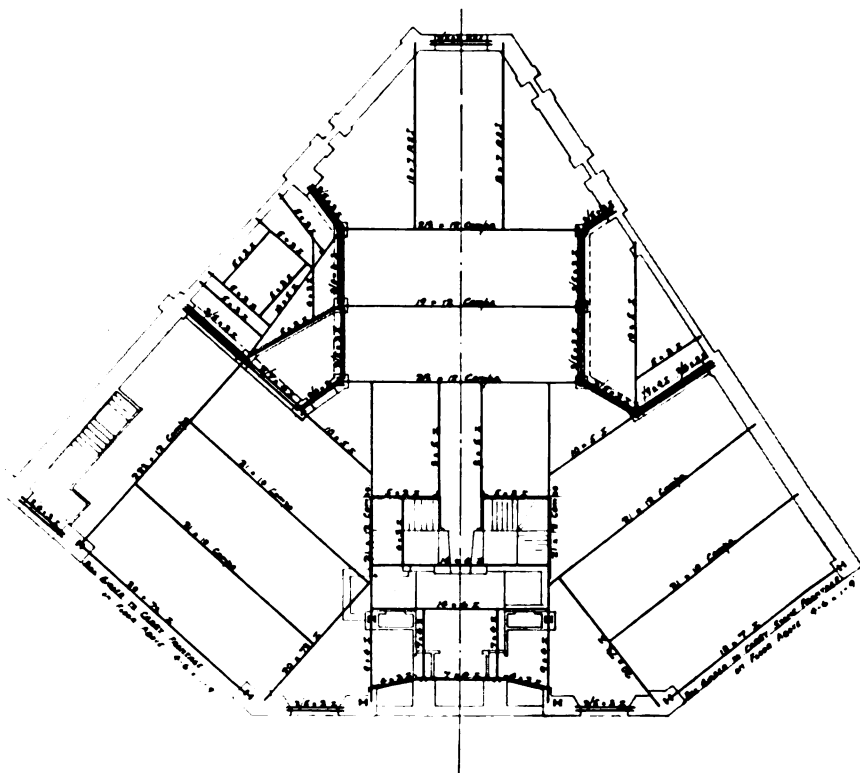


FIG. 5. Ground Floor Plan, showing arrangement of first floor steelwork.
EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.

rods of $\frac{3}{8}$ in. and $\frac{3}{4}$ in. diameter, in accordance with drawings prepared by a competent engineer.

The main staircase provided an interesting feature in the construction, this being carried out with round steel columns, cranked joist stringers, and reinforced concrete steps. The reinforced concrete work was executed with 6 in. and 5 in. landings with $\frac{1}{2}$ in. rods at about 6 in. centres and $\frac{1}{4}$ in. distribution rods at 18 in. centres, and a minimum thickness of 5 in. was employed under the tread finishing, at the intersection of tread and riser, and rods parallel to the length of the flights were placed in both upper and lower surfaces of this concrete. The rods in the two surfaces were connected

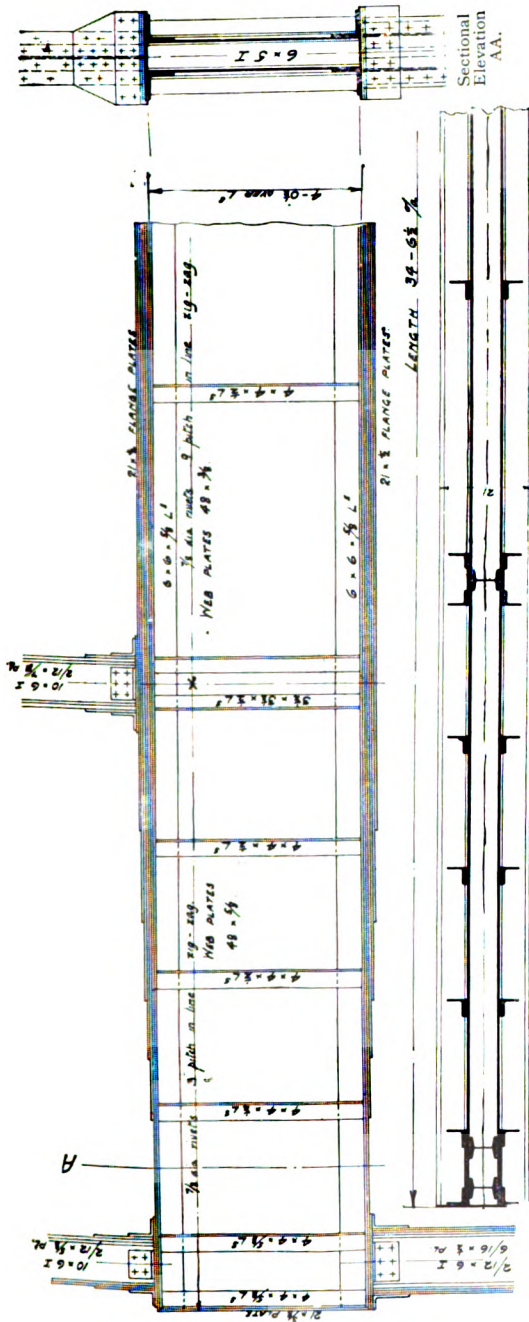


Fig. 6. Empire House, Detail of Box Girder.

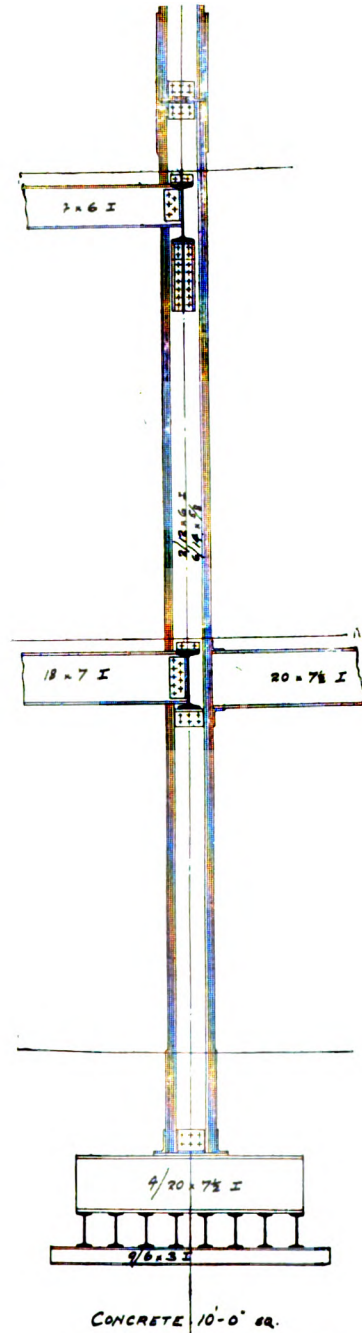


Fig. 7. Typical Stanchion Detail.

EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.

by wire binding and the ends of all bars were hooked, some hooks passing over the flanges of the steel framing. It is interesting to note that the box girders in the building weigh about 12 tons each, and approximately 600 tons of steel was used in the steel framing.

A typical framing plan showing the lay-out of the steel beams and stanchions in India House is shown in Fig. 9, and it will be seen that the method employed is similar to that described for Empire House, with steel beams having fairly long spans, necessitating in many cases the

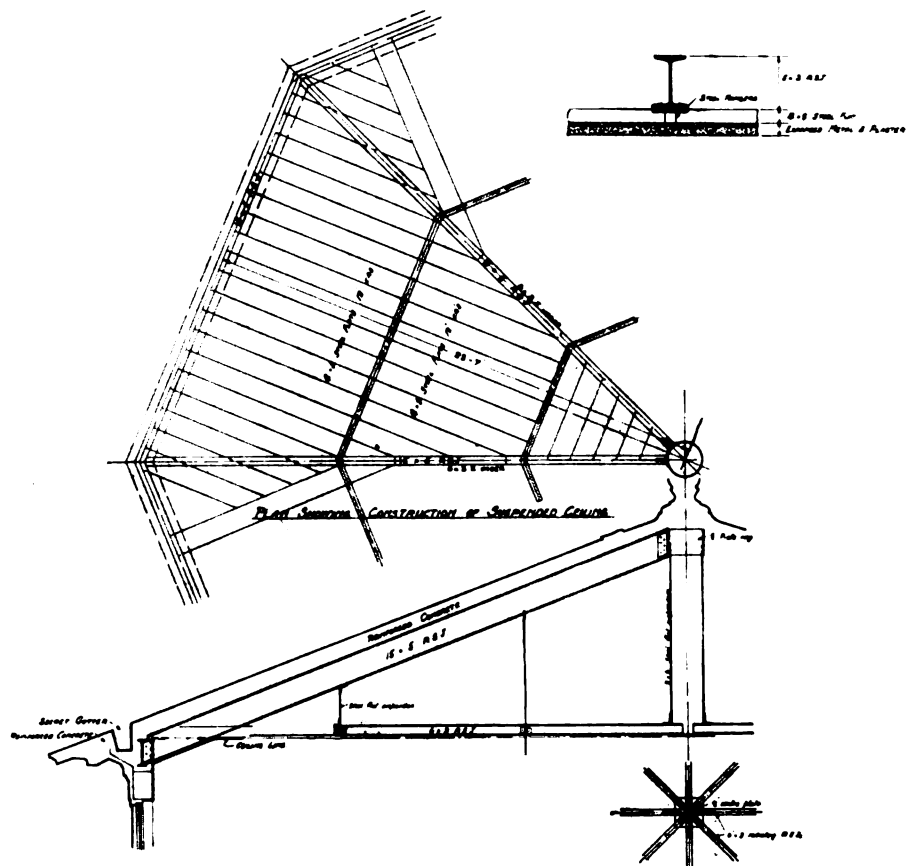


Fig. 8. Showing Detail of Dome Construction.
EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.

use of compound sections. The bays are filled in with reinforced concrete slabs, the span in one case being nearly 18 ft. The main front in this building is also carried at the second floor level by box girders, two of which are employed, each with a span of about 35 ft., and the girders have a total depth of 4 ft. 5½ in. and a width of 1 ft. 9 in. The section is built with two 48 in. by ⅜ in. web plates 10 in. apart and four 21 in. by ½ in. and one 21 in. by ⅜ in. flange plates, connected to the web with 4 in. by ¼ in. by ⅜ in. angles. Two stanchions are carried by this girder and

four angle stiffeners are provided below these and also at the ends where stanchions occur above the bearing area, and in the latter cases two short lengths of 10 in. by 8 in. rolled steel joists are placed between the web plates to transmit the load through the girder to the stanchion below. Intermediate stiffeners are also provided to the web plates, these being of tee section. The stanchions used generally are compound sections built up with one or two rolled steel joists and one, two, or three plates to each flange, according to the

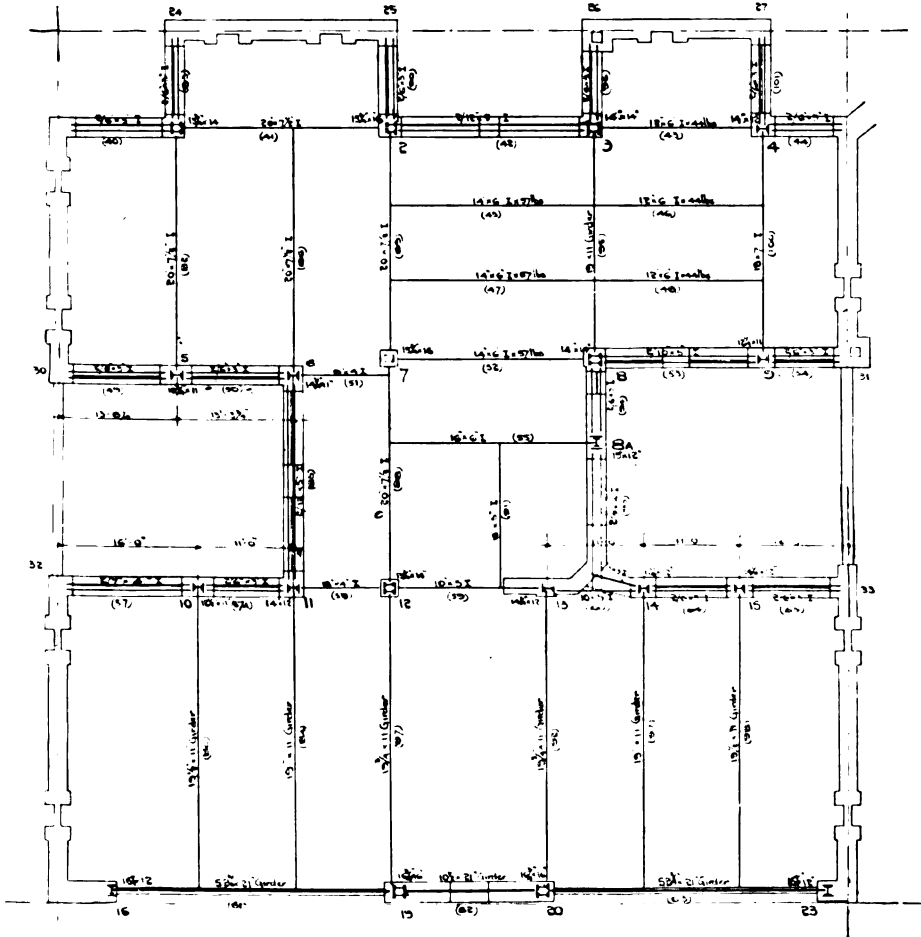


Fig. 9. India House Framing Plan.
EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.

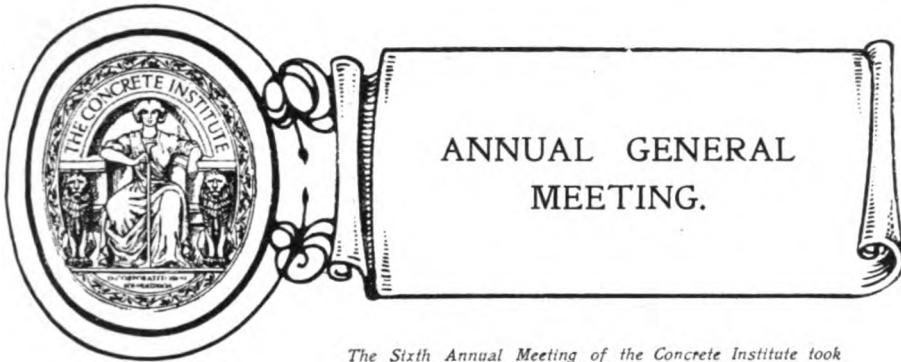
load to be carried. Grillage foundations were employed, these consisting of two layers of rolled steel joists, a typical example being composed of three 12 in. by 6 in. joists for the upper layer and ten 10 in. by 5 in. joists for the lower layer. In one instance on the ground and basement floors a solid steel column was used in lieu of a compound steel stanchion, and this was 10 in. diameter, and it extended in one length through the height of the two floors. The steel cap was 18 in. sq. and $2\frac{1}{2}$ in. thick, and the base 2 ft. 6 in. sq. and

4 in. thick, these being shrunk on to the shaft in the usual way. Connections at the intermediate floor level were required on both axes and on each side of the column, and these were arranged by providing a cast steel ring in two pieces which was bolted in a sinking $\frac{1}{2}$ in. deep, specially turned in the column for the purpose. This ring was $7\frac{1}{2}$ in. deep and it had a projecting flange 3 in. wide to provide a seating for the beams, with stiffeners immediately below the centre of the latter, and, in addition, a $\frac{1}{2}$ in. plate having a sliding fit to the shaft was introduced, to bolt the top flanges of the beams and hold them to the column. This forms a very good method of connection to a solid steel column and is much better than many of the types adopted by steel contractors.

The general contractors for the two buildings were Messrs. Wm. Taylor and Co., Lower Mall, Hammersmith. The steelwork to Empire House was executed by Messrs. Powers and Deane, Ransome's, Ltd., Cubitt Town, and that to India House by Messrs. Archibald D. Dawnay and Son, Ltd., Battersea; while the reinforced concrete staircase was carried out by The Patent Impervious Stone and Construction Co., Hammersmith.



Fig. 10. Reinforced Concrete Staircase at Empire House,
EMPIRE AND INDIA HOUSES, KINGSWAY, W.C.



The Sixth Annual Meeting of the Concrete Institute took place on May 6th, at Denton House, Vauxhall Bridge Road, when the Annual Report was presented. Below we give a Summary of the Report.—ED.

A SUMMARY OF THE REPORT OF THE COUNCIL.

Membership.—The Concrete Institute had on April 30th, 1915, 935 Members, 40 Associate Members, 8 Associates, 64 Students, 5 Special Subscribers, and 14 Honorary Members.

There has not been considerable decrease in Membership owing to the war, although a number of Members of the Institute are serving with the Forces. Names of Members serving with the Forces of whom information has been received are recorded in a Roll of Honour which is given as an appendix to the Report.

Finances.—In 1913 there was a deficit. In 1914 there has again been a deficit, but a smaller one, owing to an appeal by the former President, Mr. E. P. Wells, suggesting that Members should increase their subscriptions to the new rate of two guineas which was adopted when the total Membership of all classes had reached one thousand. The new subscription rates and entrance fees have also improved the income.

Meetings.—The extra number of general meetings has been maintained, but in view of a number of the Junior Members joining the Forces, the Educational Lectures and the Informal Meetings of Junior Members were abandoned for this Session; such educational work will be resumed in the future.

Award of Institute's Bronze Medal.—As the result of a ballot among Members of Council, the bronze medal for the best paper read in the 1913-14 Session has been awarded to Mr. W. Cyril Cocking for the paper entitled "Calculations and Details for Steel-frame Buildings from the Draughtsman's Standpoint."

Scope of the Institute.—In the previous Annual Report reference was made to the enlargement of the scope of the Institute and the alterations to the Rules, which were approved at General Meetings. It was explained that alternative policies would be placed before a General Meeting for decision. This has been deferred until the war is concluded.

The Institute and the L.C.C.—As regards the work of the Council and Committees during the past year the chief business has been the further consideration of the Regulations proposed to be made under the provision of Section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete. A revised draft of the proposed Regulations was received in September, and the Council and Committees in joint Session reconsidered the matter, and made various recommendations for further amendment. It was found that the Institute's previous suggestions had been embodied in large part in the revised draft.

Bills of Quantities.—A letter was addressed by the Quantity Surveyors' Association to the Commissioners of H.M. Office of Works and Public Buildings deprecating

the preparation by the Consulting Engineers (employed by the Department to make calculations and prepare drawings for reinforced concrete structures) of Bills of Quantities upon which competing contractors based their tenders, and advocating the extension to such work of the present system, under which quantities for buildings erected in other materials were prepared by surveyors appointed by the Department. The Council of the Concrete Institute, together with the Quantity Surveyors' Committee of the Surveyors' Institution, gave its support. The letter inquired whether a deputation from the three bodies in support of this suggestion would be received, but nothing has yet resulted.

New Members of Council.—In June, 1914, Mr. J. S. E. de Vesian and Dr. J. S. Owens were co-opted as Members of Council.

Nominations for Vacancies on other Committees.—The *Engineering Standards Committee* having asked the Institute to nominate a representative to take the place of Mr. W. G. Kirkaldy as representative of the Concrete Institute on the Sectional Committee on Bridges and General Building Construction, the Council appointed Mr. R. H. Harry Stanger in that capacity.

The Council also nominated Mr. H. Kempton Dyson, Secretary of the Concrete Institute, to fill the vacancy created by the death of Mr. Kirkaldy upon the *Joint Committee on Reinforced Concrete* conducted by the Royal Institute of British Architects.

The Institute's Examinations.—The Rules and Syllabus of the proposed examination of the Concrete Institute were appended to the previous Report of the Council. The Council has decided that the first examination shall be held early in June, 1916, of which due notice will be given.

Members Deceased.—The Council regrets to record the decease of:—Hon. Members: Monsieur Armand Considère and Monsieur Edmond Coignet. Member, killed in action: Major A. H. Tyler, R.E., F.R.G.S. Member: Mr. F. Dare Clapham, F.R.I.B.A.

Library.—Several donations to the Library have been received by the Council from authors, publishers, and kindred societies, and the Council expresses thanks to the donors. A list of books received is published from time to time in the *TRANSACTIONS*.

Finance and General Purposes Committee.—The Finance and General Purposes Committee has held regular meetings preliminary to each Council Meeting, and the general results of their deliberations are contained in the foregoing particulars of the Council's work for the year.

Science Standing Committee.—In addition to considering the L.C.C. Regulations for Reinforced Concrete, the Science Standing Committee has been concerned, jointly with the Reinforced Concrete Practice Standing Committee, in the compilation of a Standard Specification for Reinforced Concrete Work which was submitted in draft for discussion at a General Meeting. The Report has yet to be revised in the light of the discussion before it can be re-submitted to the Council for their final approval.

The Science Standing Committee has the following matters under consideration:—

1. Standardisation of joints and connections in reinforced concrete.
2. Amendment of the Standard Specification for cement.
3. Co-ordination of the Standard Specification for structural steel of all kinds.
4. The adhesion of and friction between concrete and steel.
5. Reinforced concrete piles.
6. The effect of sewage upon concrete.
7. The effect of oils and fats on concrete.

Reinforced Concrete Practice Standing Committee.—During the past Session the Reinforced Concrete Practice Standing Committee has met, in conjunction with the other Standing Committees and the Council, to consider the L.C.C. Regulations for Reinforced Concrete.

The Committee has held joint meetings with delegates of the Quantity Surveyors' Association and with Members of the Concrete Institute who are Quantity Surveyors by profession. The Joint Committee in the previous Session submitted for discussion at a General Meeting a Draft Report on a Standard Method of Measurement for Reinforced Concrete. During the past Session a number of meetings have been held for the purpose of revising this Report, delegates attending from the Royal Institute of British Architects, the Surveyors' Institution, the Institute of Builders, and the National Federation of Building Trades Employers of Great Britain and Ireland. As the outcome a final Report upon the Measurement of Reinforced Concrete in Building Construction was submitted to the Councils of the Concrete Institute and the Quantity Surveyors' Association and adopted by them. It has been arranged to continue the work of the Committee with a view to the formulation of a Report on the Measurement of Reinforced Concrete in Engineering Works as distinct from Building Works. At the same time as the Draft Report above referred to was submitted for discussion, another Draft Report of the Reinforced Concrete Practice Standing Committee alone was put forward. The work of revising this Report has now to be undertaken. The final Report will contain suggestions as to the manner in which engineers should furnish information to quantity surveyors, the nomenclature to be used, and a tabulated form for preparing quantities for reinforced concrete.

The Committee has also been engaged jointly with the Science Standing Committee in the preparation of the Standard Specification for Reinforced Concrete as recorded above.

The Reinforced Concrete Practice Standing Committee has the following matters under consideration :—

1. Advice to clerks of works, inspectors, and foremen as to methods of properly executing concrete and reinforced concrete work and of preventing defects and failures.
2. Regulations, recommendations of joint committees, and various methods of calculation in respect to the design of reinforced concrete and the like.
3. Forms and centering for reinforced concrete work.
4. Standard concrete mixtures for general purposes.
5. The use of cinder, ash, clinker, and breeze in concrete.
6. Means of keeping reinforcements in place when concreting.
7. Methods of making concrete watertight and of waterproofing concrete.

Tests Standing Committee.—The Tests Standing Committee has held joint meetings with the Council and the other Standing Committees, as previously mentioned.

The Tests Standing Committee has the following matters under consideration :—

1. The effect of the presence of sulphur and its compounds in aggregates.
2. The grading of aggregates.
3. The effect on concrete of physical changes: (1) Effect of temperature; (2) Effect of moisture contents
4. The effect of the use of sodium silicate on the surface of concrete as affecting reinforcing metal.
5. The erratic results obtained by the Vicat needle in ascertaining the initial setting time of cement.
6. The collection of data regarding the elastic moduli of concrete for stresses within working limits.

Parliamentary Standing Committee.—The Parliamentary Standing Committee has held joint meetings with the Council and the other Standing Committees, as previously mentioned.

It has the following matter under consideration :—The draft of a Bill promoted by the Society of Architects for the registration of architects.

Investigation Committee.—The Investigation Committee has had under consideration reports of failures on reinforced concrete structures, but as the information

contributed was confidential, the results of their deliberations cannot be furnished in the form of a Report.

Joint Committee on Loads on Highway Bridges.—The Joint Committee on Loads on Highway Bridges conducted by the Concrete Institute will shortly meet to consider their final draft Report. It is intended to be presented for discussion at a General Meeting next Session.

THE ANNUAL GENERAL MEETING.

The Annual General Meeting of the Institute was held at Denison House, Vauxhall Bridge Road, S.W., with the President, Professor Henry Adams, in the chair. A short discussion followed the presentation of the Report, and of this discussion we give a summary below.

A special feature of the meeting was the presentation of the Institute's medal to Mr. W. Cyril Cocking for his very excellent Paper entitled "Calculations and Details for Steel Frame Buildings from the Draughtsman's Point of View."

DISCUSSION.

THE PRESIDENT, in moving the adoption of the Report and Accounts, observed that, in spite of all difficulties, the membership had not decreased, but had still continued to advance during the last three years. It was a matter for congratulation that there had not been a considerable decrease of membership owing to the war, although a number of members of the Institute were serving with the Forces. The Roll of Honour was not yet complete, and they would be glad to receive further information. The list of Papers read during the session had been the longest the Institute had ever had. As regarded the work of the Council in Committees, they would be glad to hear that the Reinforced Concrete Regulations of the London County Council had at last passed through all their stages of revision, which had occupied some years, and were now in the hands of the Local Government Board and the London County Council for presentation to the Council and ultimately to the public. They were very well pleased with the result, and when they saw the document he thought the members of the Institute also would be satisfied. It did not profess to be perfect, but it was as perfect as the combined effort of all the various institutions concerned could make it, and the London County Council had met them in a very fair and friendly way with regard to their suggestions.

As regarded the Balance Sheet and Expenditure Account, they would notice balance, being excess of expenditure over income, £127 16s. 6d. They had put by a sum of £150 towards their printing account for the next session because the printing had been somewhat delayed owing to the war. The printers said that so large a number of their staff had joined the Forces that they were unable to use their ordinary expedition, but the Council had provided for that in their future accounts.

The various Committees had been very active during the session. They had still in hand many items which would perhaps remain in hand for some time because they were gathering information, and if members could assist in any way in supplying information, the Council would be very glad indeed to receive it.

MR. MORGAN E. YEATMAN, M.A., M.Inst.C.E., seconded, and the motion having been put to the meeting, the Report and Balance Sheet were unanimously adopted.

PRESENTATION OF INSTITUTE'S MEDAL.

THE PRESIDENT said it was the custom to present the Medal of the Concrete Institute to the author of the best Paper which had been read during the previous session. Where so many good Papers were received it was a matter of considerable difficulty to select one that was pre-eminent, but in the present instance he thought they would agree that the selected Paper was an uncommonly good one. To members practising in different branches different Papers would appeal in a different manner. The Paper by Mr. W. Cyril Cocking, entitled "Calculations and Details for Steel-frame

Buildings from the Draughtsman's Standpoint," received the highest number of votes of the Council, and he had very much pleasure in asking Mr. Cocking's acceptance of the Institute's Medal. (Cheers.)

MR. W. CYRIL COCKING, M.C.I., in accepting the honour, said he should treasure the Medal as a lively memory of the privilege they had accorded him in allowing him to address them, and the still greater privilege of having his Paper discussed there by engineers and authors who had been successful in attaining the highest positions in their branch of the engineering profession.

He then went on to make a most interesting and important speech, the report of which will be found in our Editorial Notes on page 269.

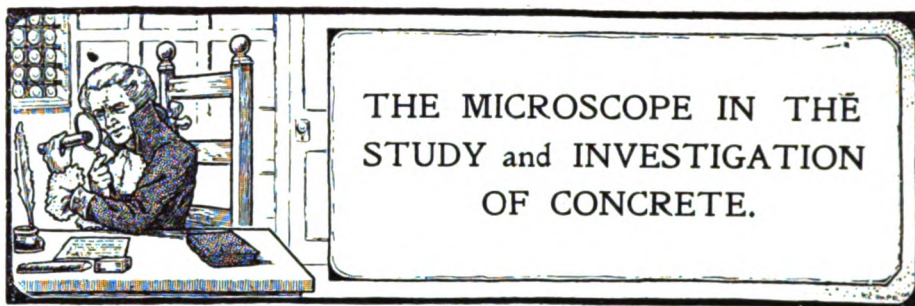
MR. HENRY J. TINGLE, M.Inst.C.E., M.C.I., proposed a vote of thanks to the President for the arduous work he had carried out during the year on their behalf. He headed the list of attendances at Council and Committee Meetings, and his work there was very valuable from his experience and tact.

MR. S. BYLANDER, Past President, J.I.E., M.C.I., in seconding, observed that the Council during the past year had carried through several important improvements as regarded rules, specifications, and other matters for the benefit of the profession, and he hoped that for many years the President would be able to lead and guide them in the proper performance of their duties as constructional engineers.

The resolution was unanimously adopted.

THE PRESIDENT, in acknowledging the compliment, said he did not take it so much as a personal vote to himself for the small services he had been able to render as to the Council and the officers of the Institute. The work was not individual but collective, and in that its value consisted.

The proceedings then terminated.



By NATHAN C. JOHNSON,

Engineer of Tests, Raymond Concrete Pile Company, New York.

The articles which we are reproducing on the use of the microscope in the study of concrete are of considerable interest, but in reprinting them we would state that the opinions expressed are those of the author, and the Journal as such does not necessarily associate itself with the conclusions arrived at. The articles are reprinted by the courtesy of the "Engineering Record," U.S.A., and the illustrations have been placed at our disposal by the author.—ED.

THE possibilities of the microscope as a tool for the concrete engineer—such is the keynote of the present series. In view of the service the microscope has rendered the metallurgist, it occurred to the writer that it would be equally valuable in disclosing the real character of concrete. Concrete, no less than steel, is dependent for mass perfection upon the perfection of its smallest parts, and in no way so far known can the quality of these small components be so well, so quickly, and so accurately determined as by the use of the microscope.

GENERAL CONSIDERATIONS.

There is special reason, moreover, why every means for studying concrete should be investigated, for no other construction material to-day, not excepting steel, has so widespread a use and enjoys so great a popularity. Its possibilities are limitless. Its applications are numberless. Its making and forming are of the simplest. Some roughly measured stone and sand from a convenient bank, a little cement from a near-by dealer, some water, a little turning with shovels or by machine, a board form, and the trick is done. This is essentially true whether the work be great or small. From an economic standpoint, whether the economist be witting or unwitting, professional or lay, skilled or unskilled, educated or ignorant, such an appeal is irresistible.

But desirable as it is to have so easily made, so adaptable, and so cheap a material within the reach of everyone, those very qualities have led to its abuse; and with abuse inevitably has come failure. It is a peculiarity of Portland cement-

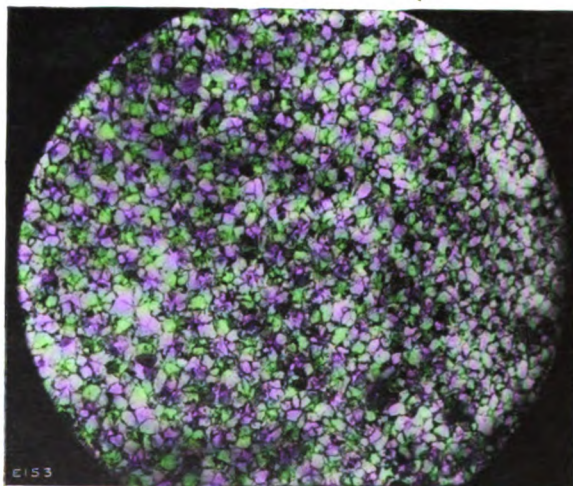


Fig. 1. Sandstone Grain magnified 40 diameters.
THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

concrete that a fair degree of success is not prevented by a great deal of bad treatment in making, and that concrete made by bad treatment looks almost as well—at the start—as that which is properly made. It takes time, oftentimes years, to reveal the hidden weakness, but time will surely bring it to light. There have been

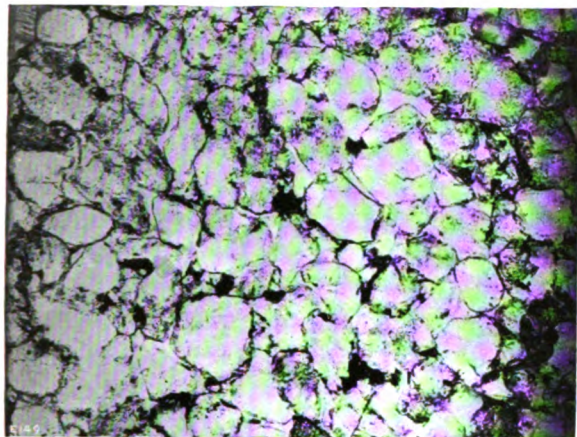


Fig. 2. Enlargement of Sandstone Grain shown in Fig. 1.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

some unfortunate failures in the past, but there are more to come in the future, and these will give Portland cement and Portland cement-concrete an undeserved reputation for unreliability, unless it can be shown why these failures occur, how they may be prevented, and in what ways our present methods or materials must be changed to make concrete "as enduring as the living rock."

ing attendant upon usual methods of mixing and placing would make microscopical examination of little value, so far as actual construction operations are concerned, especially as the examination is made of samples of restricted size. The same objection might be raised with equal justice in regard to the examination of steel. It is impossible to examine the whole of one ingot, or the whole of every ingot from each heat; but by taking representative samples from each ingot, or from each bloom, and thus knowing the quality of the product, with this information and a knowledge of the raw materials—the ore, or pig-iron, the flux, fuel, etc.—that entered into it, very close control is possible on huge-scale operations.

It is the belief of the writer that the same will prove true of the microscopic examination of concretes. The method is so simple, so direct, the results so indisputable and convincing, that it should make a very powerful appeal to everyone interested in the concrete industry. Nor is the apparatus required necessarily elaborate or expensive. It should further be remembered that in this work light is reflected from a single polished surface, which greatly simplifies the procedure. It is neither advisable nor necessary to grind down the section until

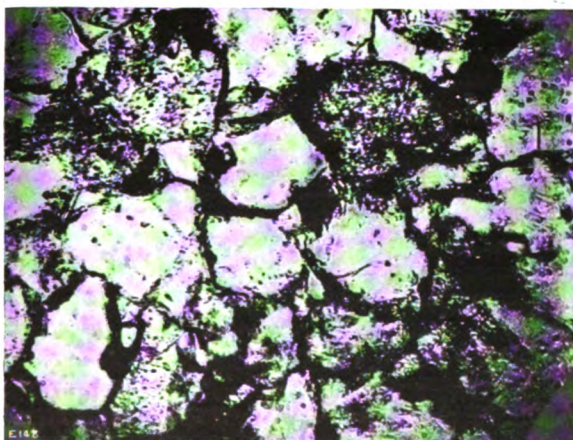


Fig. 3. Enlargement of Sandstone Grain shown in Fig. 1.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

light can be transmitted through it, as has been required in most of the microscopic work done heretofore on cements and rocks.

It is impossible, in a few lines, to do more than to indicate some of the possibilities of microscopic examination in concrete work—the quantitative analysis of concretes; the detection of impurities, such as vegetable, adhesive clay or other films on the sand or stone; impurities and adulterants in the cement. Although this article and those following are not primarily intended as an exposition of microscopic investigation, a few illustrations of such uses will be given and a few points touched upon which, it is hoped, will make a direct appeal to all who are interested in obtaining better concrete. It is hoped, also, that this appeal may prove general, and that microscopic examination will not be confined to the few whose names are already well known, but may soon find its way into every cement laboratory worthy of the name.

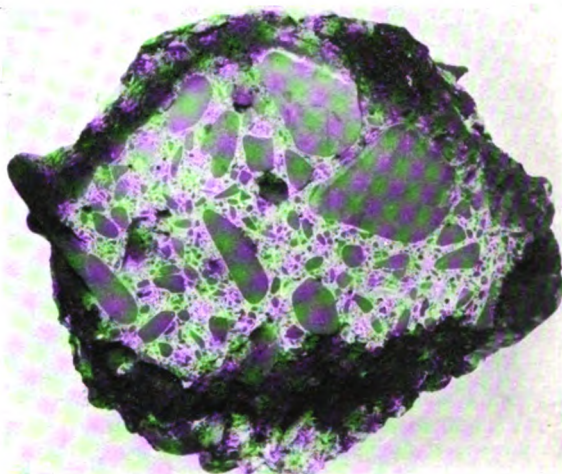


Fig. 4. Concrete from Wing Wall of Dam at Cornell University.
THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

PRELIMINARY CONSIDERATIONS.

Before taking up the detailed study a review of what is well known in regard to the general nature of concretes and their making may prove of value. In this review it will simplify matters to put aside for the present all chemical questions in regard to the nature or the manufacture of cement and to assume that the cement is a perfect product which forms, when mixed with water, a sort of "mineral glue" which coats the sand grains and the stones so that they stick together. In other words, take the name at face value—as "cement."

There are four substances which enter essentially into concrete—stone (crushed stone or gravel), sand (either natural sand or crushed rock), cement, and water.

These four substances are necessarily variables. Each has certain inherent properties which must be taken into account in forming the mass. If cement and water are considered as one substance—i.e., as a glue—the number of variables can be taken as three. What relation, then, do these substances bear one to another? And what function does each perform in the concrete?

The first requisite in concrete is strength, whether tensile, compressive or shearing. The second requisite is density, but experience shows that density and strength are correlative. This is true of natural stones as well as of concretes (tests by R. L. Humphrey, *Engineering and Mining Journal*, June 20th, 1902, page 921). Obviously, also, the strength of the composite product cannot exceed the strength and density of the strongest and densest of the materials.

STRUCTURE OF STONE.

Assume for convenience that sandstone is selected as the coarse aggregate and sand of like nature as the fine aggregate. The structure of a sandstone sand grain,

as seen through the microscope, is shown in *Fig. 1*. As a piece of the coarse aggregate exhibits throughout an exactly similar structure, it is fair to take the ground that all of the pieces of stone that go into the concrete are built up of tiny particles in the same way. Further, it is found that any part of a larger mass of the stone, such as a test cube, shows the same structure throughout as the sand grain, so that the strength per unit area inherent in the sand grain should be the same as that inherent in the test cube, which crushed at 12,200 lb. per square inch. Therefore, considering for the sake of simplicity that all stresses in the concrete are compressive,

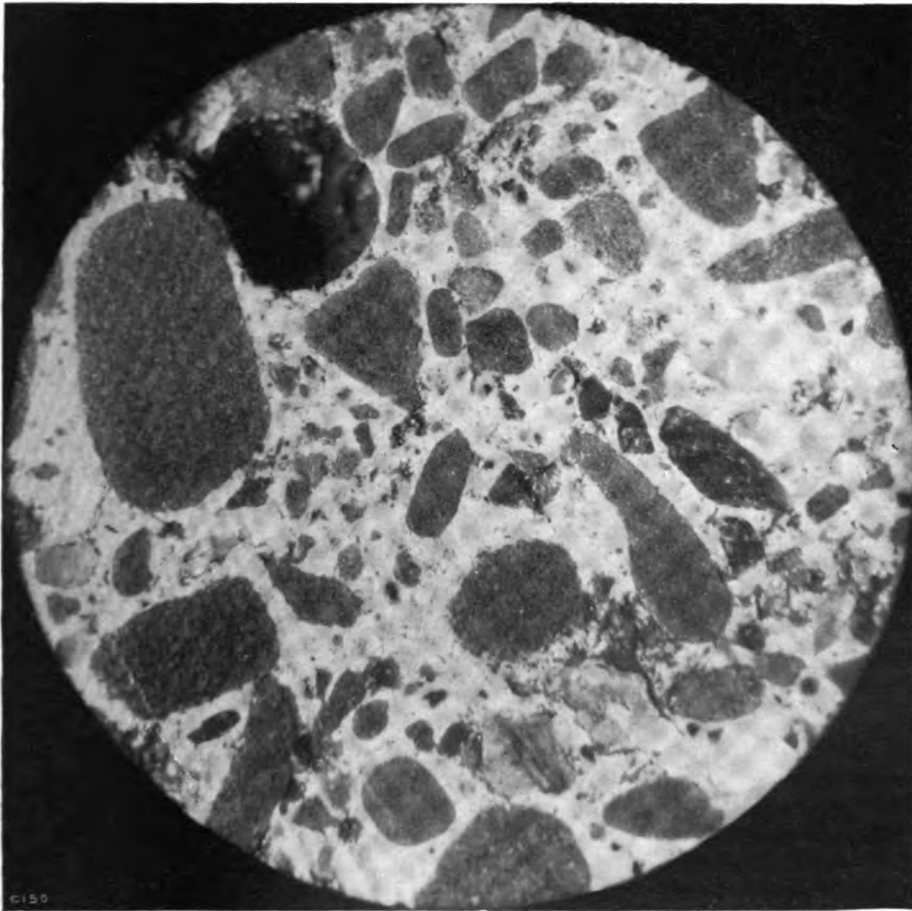


Fig. 5. Photo-Micrograph of Mortar taken from Specimen shown in Fig. 4.
THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

if a concrete is made of this stone and this sand, any failure to withstand a crushing stress of 12,000 lb. per square inch must be ascribed either to the properties, the quality, or the improper use of the cementing material which holds the sand particles and the stone particles together, or else to the presence of some foreign substance in the matrix in which the particles are embedded.

CONCRETE SHOULD APPROACH STONE IN DENSITY.

It is axiomatic and also corollary to what has been before stated, that the more nearly a concrete approaches natural stone in density, the better and more economical

will it be. If Fig. 1 is examined closely it will at once be seen that the sandstone sand grain is, of itself, a concrete, that it consists of tiny particles, closely compacted, held in place by, and filled between with, some cementitious substance, which is known to be either iron or silica, or both. Magnifying this same spot to a higher degree, as in Figs. 2 and 3, the similarity between this natural and artificial concrete is made even clearer. But there is one marked difference, which is very evident—*i.e.*, in natural stone the component particles are packed far more closely than are those of artificial concretes, and the cementitious layer between them is far thinner than the layer obtained by artificial means. This is a significant point and should be remembered. Considering a hypothetical case, assume that 1 : 2 : 4 sandstone concrete in pure compression breaks at 2,200 lb. per square inch at a mature age, say one year. The cement alone should not crush below 8,500 lb. per square inch, nor the sand or stone below 12,200 lb. per square inch. To the eye the concrete appears perfect. Some tiny airholes may be visible, but these could hardly account for the comparatively low strength. The concrete may be struck with a hammer, or pulverised in a mortar, or analysed chemically, and the relatively low strength attributed to "water-worn sand," or "dirty sand," or "poor cement," or any one of a hundred commonly-assigned possibilities. The problem is a serious one, *for so far as the strength shown by the concrete is concerned, we might better have used shale, or some waste rock that would crush at not less than 2,200 lb. per square inch*, instead of the expensive stone. Further, this strength of 2,200 lb. per square inch is in considerable excess of the average strength of concrete made under field conditions, so that, so far as concerns the strength of the product that is being turned out by hundreds of thousands of yards daily, an even poorer grade of stone might have been used, with a tremendous saving to contractor and owner in cost of aggregate.

Extensive tests, however, would seem to disprove the correctness of the foregoing reasoning. Under certain conditions the ultimate strength of a concrete may equal the strength of the stone aggregate (Taylor and Thompson, *Concrete Plain and Reinforced*, pages 391 and 392), though this is by no means true of all concretes. However, wide experience leads to the belief that a strong aggregate makes a strong concrete, and a weak aggregate a weak concrete. Theory and practice are seemingly antagonistic, but it may be that this established fact and the conclusions of the foregoing paragraph will after all be found to agree if the nature and structure of concretes be studied carefully.

IDEAL IN PROPORTIONING.

The theoretical ideal in proportioning materials for concrete is to approach maximum density—*i.e.*, the density of the large aggregate—by putting in just enough sand to fill the spaces between the stones, and sufficient cement to coat over the entire surface of the stones and of the sand grains, as well as to fill the spaces between the sand grains. The exact volumes of these stone-voids and sand-voids are often determined for practical work with the greatest care, but when concrete is proportioned on the basis of such determinations, something seems to go awry. The concrete does not show maximum density, as was planned; oftentimes far from it. Distrust of proportioning on the basis of void determinations has been growing rapidly of recent years. Evidently, either the ideal is wrong, or the method of attaining it is poorly chosen.

It may help in understanding these matters to look at the inside of concrete of known composition, made by approved field methods. The specimen shown in Fig. 4 was taken from the wing wall of a dam at Ithaca, N.Y., this dam being part of the hydraulic power development of Cornell University in Fall Creek Gorge. Unusual care was taken in proportioning, mixing, and placing this concrete, in order to have it water-tight, so that the specimen may be considered a representative product of accepted field practice.

The specimen was prepared by grinding until a smooth surface was obtained. As the plane of this surface would have laid about 8 in. from the outer face of the wing wall, its features should be characteristic of the entire structure. As is usual, when void determinations were made on this stone, it was shaken down, in order to compact it as closely as possible. Evidently, in the present relative positions

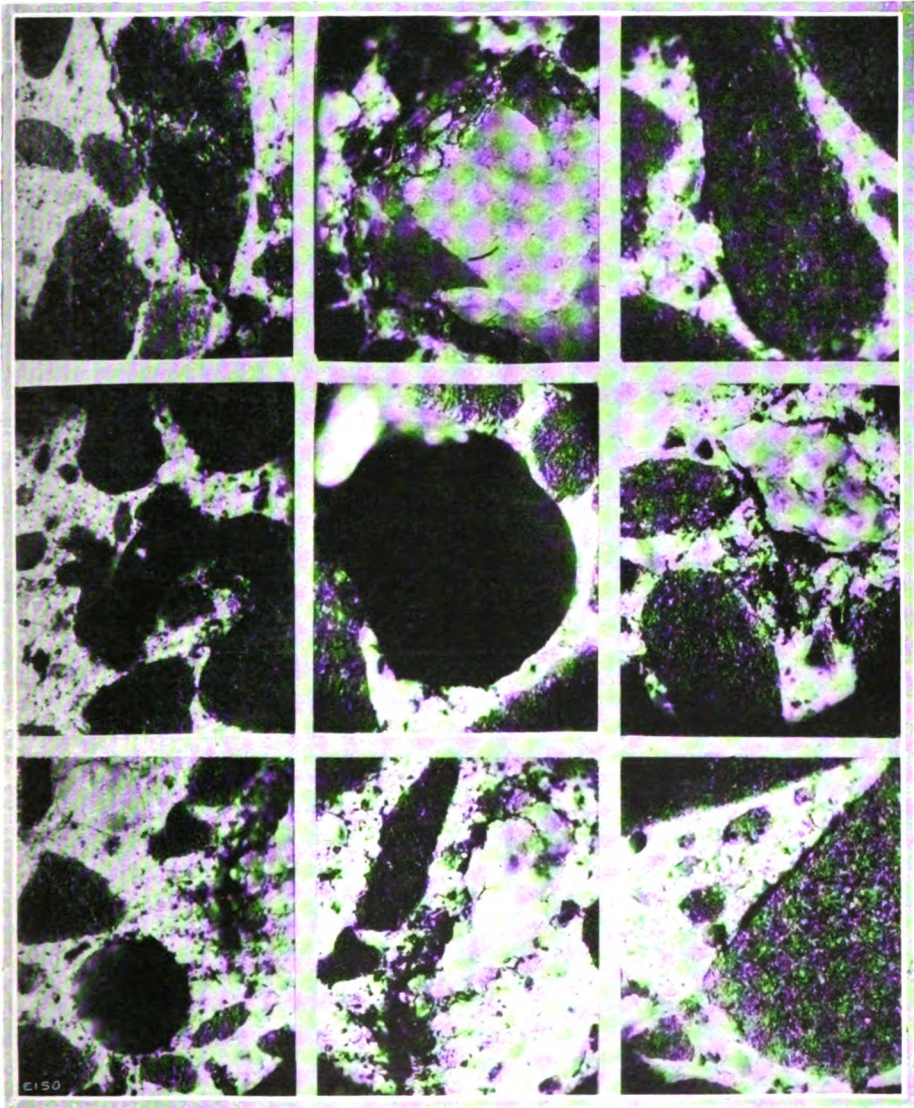


Fig. 6. Microscopic Traverse of Matrix of Concrete Specimen shown in Fig. 5.
THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

of the stones, the percentage of voids would be enormously increased. This would also be true, though to a less degree, even if the stones had not been shaken down. If this same increase of voids holds for the sand, the unreliability of void determinations as a basis of proportioning is readily understood.

AIR HOLES.

The next most striking feature of this surface is the number and size of air holes visible. Some of them are very small, so small as to seem negligible; while others are of appreciable size. One or two lie directly adjacent to pieces of stone. If the surface were magnified a hundred times it would be a field 100 ft. on a side, with each stone a boulder and each air hole a cavern. Further, these large air voids (and the small ones as well) are of indefinite size and depth below the surface, so that conditions may be better, or may be worse, in other portions. Evidently, however, the matrix of sand and cement in which the stone is embedded is of uncertain texture. As this matrix is relied upon, not only to hold the stones together against disruptive stresses, but also to transmit compressive and other stresses from one stone to another (it being remembered that the stone has the greatest density and greatest strength of all the constituents of the mass), it is evident that some or all of these stones will be called upon to bridge air gaps of greater or less size and extent.

It requires little or no imagination to picture the result of an application of stress to such a mass. Unless the large aggregate is very strong indeed, local failure must result at comparatively low unit stress values. Further, even if the large aggregate be very strong, low unit stresses on the mass may result in extremely high stresses on unsupported units, with local failure again resulting. One local failure brings other local failures, like a string of dominoes, until the whole mass gives way. It begins to be evident why rock of low strength cannot successfully be used as aggregate, unless conditions of mixing and placing are superior to those that obtained during the making of the wing wall from which this sample was taken.

But the tale has not yet been fully told. It has been seen that the matrix of sand and cement in which the stones are embedded has to transmit stresses. This being true, it follows that the sand grains, as components of that matrix, must each bear a share of such stresses. In the matrix of this specimen are they bearing the load of which they are inherently capable? Or must they, too, bridge crevasses and pits, as do the stones? The microscope gives the answer.

CHARACTER AND STRUCTURE SAME THROUGHOUT MASS.

Fig. 5 is an enlarged photo-micrograph showing a piece of the mortar taken from between the stones of the specimen shown in *Fig. 4*. This is a characteristic picture of the sand and cement matrix of the average concrete. The photograph was obtained by attaching a camera to the eye end of a microscope and enlarging the negative obtained, so that the field as seen by the eye is magnified about 25 diameters. In this way the large sand grains are seen as large as the pieces of stone used in the concrete, the small sand grains are of relatively large size, and minute holes, too small to be observed by the unaided eye, appear almost as caverns.

Anyone not accustomed to microscopic work would be unwilling to believe that this figure was other than a direct photograph of a surfaced concrete. There is the same characteristic structure, the same "stones," the same wide separation of aggregate which showed previously why void determinations on stone are unreliable as a basis of proportioning concrete. But perhaps the most puzzling of all are the dark masses and particles lying between the sand grains, too small to be sand and also of a different colour. Investigation proves them to be unhydrated cement. If it has ever been wondered why it was possible to grind up a set concrete and obtain a re-set, that wonder now disappears. This unhydrated cement—often as much as 75 per cent. of the amount used—lies inert in the matrix, performing no function that might not be better performed by as much fine sand. Evidently the mixing processes are very inefficient. Nor is this an isolated case. No concrete that has yet come under the writer's observation fails to show a large percentage of unhydrated

cement. Some of these concretes are more than thirty years old, so that time cannot be said to be the sole determining factor in this regard.

TRAVERSE OF THE MATRIX.

The specimen of mortar taken for this micro-examination was cut at random from the chunk shown in *Fig. 4*. Further, this specimen was surfaced at random, the surface thus obtained being polished with rouge to give proper relief to the various constituents. In area it did not exceed $\frac{1}{2}$ in. on a side, but the character of matrix found in this area is shown in the nine photographs of *Fig. 6*. Many more such photographs could have been obtained within this area, but these nine should be sufficiently horrible examples.

With a view to illustrating the working theory before stated—i.e., assuming pure compressive stress and the bridging of air gaps by aggregate—these photographs have been arranged in such position as to make the effect of a vertical stress most evident. Each piece of aggregate in these pictures is a grain of sand, the magnification being about sixty diameters. Many of these sand grains show in *Fig. 5* as well, where they can readily be identified by their shape and their relation to other particles determined. Air voids are readily distinguishable, as they show dead-black and structureless.

In *Fig. 6*, View 1 shows a sandstone sand grain with an air gap along its lower side and half-way around one end. Contact with matrix is had only on the upper side and at the end not shown.

View 2 shows a quartz sand grain almost completely isolated by air gaps and surrounded on all sides by a honeycomb of air holes.

View 3 shows a sandstone sand grain with a gap below and honeycomb matrix on all sides.

View 4 shows, in an exceptionally beautiful photograph, a quartz sand grain in almost complete isolation.

View 5 shows the large air hole seen in *Fig. 5*. As a support to even the best aggregate, its value may be questioned.

View 6 shows a quartz sand grain in almost complete isolation.

View 7 shows a limestone sand grain, with a crevasse below and a large air hole at the right-hand end.

View 8 shows a large air hole and some smaller air holes in the cement matrix.

View 9 shows an end of the quartz sand grain and the crevasse seen in View 6, with a large air hole below in the cement matrix.

TESTIMONY OF THE PHOTOGRAPHS.

Evidently, from the testimony of these photographs, the cement of concrete not only does not coat the sand and the stone, so that they will stick together, but the sand does not closely fill in between the stones so that they will have proper support. Further, the cement does not fill in between the sand grains, so that they, in turn, will be firmly bedded and be able to withstand the load. Instead of a compact substance, then, concrete is a spongy mass, from one-eighth to one-quarter of which is commonly air and water voids. Yet this specimen under examination was carefully made and placed. What careless field conditions produce is a question worth investigation.

If the steel sold for structural purposes were as spongy and of as uncertain texture as the average structural concrete, steel structural work would be twenty years behind the present mark. The unwisdom of using great masses of any material of low and uncertain strength, in place of smaller sections of proper strength, needs no demonstration.

Yet, if it were desired to aerate thoroughly any fluid, or semi-fluid, substance,

no better means could be devised than the present systems of mixing concrete by agitating, churning, and pouring, with oftentimes a drop from a considerable height into the form to make undesirable assurance doubly sure.

SUMMARY.

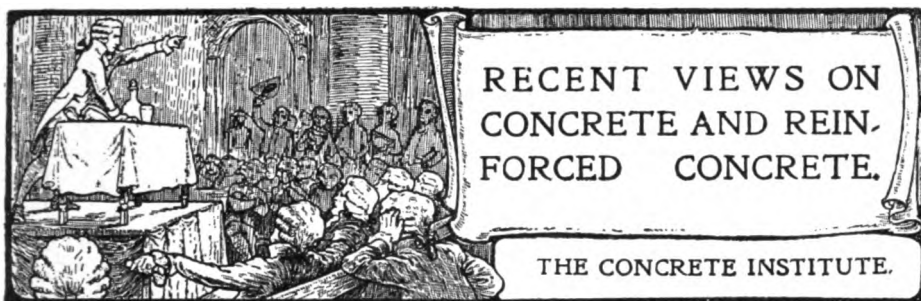
Summarising the previous discussion, it should be noted :

1. That the strength and density of concrete are in direct ratio one to another; and that for economic reasons, both direct and indirect, concrete should approach the density of natural stone.
2. That the strength and density of any concrete have, as upper limits, the strength and density of the strongest component, usually the coarse aggregate.
3. That the strength and density of any concrete are actually limited by the strength and density of the weakest component, usually the cement matrix.
4. That the cement matrix is not worked at its maximum efficiency, largely because of air and water voids in the mass.
5. That only a small percentage of the cement is hydrated, so that it can perform an active function, the remainder lying inert.
6. That this relatively small percentage of active cement is to a large extent prevented from performing its function of bonding with the aggregate by intrusive films or bubbles of air.
7. That the pieces of large aggregate and the particles of small aggregate in a concrete are in direct ratio one to another at distances approximately equally great, relative to their size; that they are not closely compacted after mixing, as they were when dry, so that void determinations are practically valueless as a basis of proportioning.
8. That because of this wide separation of both large and small aggregate, because of the imperfect contact between cement and aggregate, and because of the weakness of the cement matrix in which the aggregates are embedded, costly aggregate of superior strength must be used, instead of cheap aggregate of a strength equal to the strength of the cement matrix, in order that these weak spots may be bridged and progressive local failure prevented at low unit stresses.
9. That even with the very best of materials only concrete of inferior strength is commonly produced.
10. That present methods of mixing tend to aggravate these undesirable conditions.

The preceding discussion has dealt with certain defects in concrete due to entrapped air, but it should not by any means be assumed that entrapped air is the cause of all weak or defective concrete.

On the other hand, it should not be assumed that those weaknesses already pointed out are the only ones traceable directly to that cause. Secondary ills follow the primary weakness in an almost bewildering sequence; and in subsequent papers on microscopic study, which will appear in these columns, an endeavour will be made to touch upon a few of the most important and to indicate means for overcoming them.

The work described in these articles was first undertaken about three years ago at Sibley College, Cornell University; and during the past year has been prosecuted under an Industrial Fellowship established by the Raymond Concrete Pile Company. The writer takes this occasion to acknowledge his indebtedness to the officers and staff of Sibley College and other colleges of the university, both for permitting extensive use of apparatus and equipment and for personal encouragement and advice. The author also extends his thanks to the many individuals throughout the country who have responded, at much trouble to themselves, to requests for samples of concrete from various structures, without which this research would have been greatly restricted.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

THE CONCRETE INSTITUTE.

THE LONDON BUILDING ACTS, WITH SOME SUGGESTED AMENDMENTS

By OSBORN C. HILLS, F.R.I.B.A., District Surveyor for the Strand.

The following is an Abstract from a Paper read at the fifty-ninth ordinary general meeting of the Concrete Institute. A lengthy discussion followed, and was continued at subsequent meetings, and we also give a short report of this discussion.

THIS Paper is submitted with the view and the hope that a new Act will shortly be prepared embodying the existing Laws, By-laws, and Regulations. In 1894 our present London Building Act was passed, and thereby cancelled some thirteen existing Acts which had to be read together. Twenty-one years have elapsed since then; many decisions have been given in the High Courts, some of which completely annul the intention of the Act; many amendments have been framed, some of which have become law; and it will soon be necessary to repeat the process of 1894 and pass a new Act embodying all the existing laws and regulations, with certain desired modifications and improvements. Such improvements should be the carefully-considered results of the various authorities: the Royal Institute of British Architects, the London Society, the Town Planning Association, architects, surveyors, structural engineers, builders, and property-owners should now unite in preparing such improvements in the Acts as they consider essential in readiness for submission to the proper authority. The London County Council "fathered" the Act of 1894 through Parliament, and the writer believes they have already collected much material for a proposed new Act.

It is not suggested that the Concrete Institute should take the initiative in the matter, but, having already been recognised by Parliament in the 1909 Amendment Act, it should be prepared at the right time to suggest such amendments as will conduce towards sound construction on economical lines.

The Act should be clearly understood by all who read with average intelligence. It has to be administered by the London County Council and district surveyors. There are two important ways in which the Act needs amendment: (1) by making clear certain ambiguous and faulty wording, or wording that has been the subject of ambiguous or unsatisfactory decisions; and (2) by adding to or deducting from the existing provisions and requirements.

The chief objects of the Acts are fourfold: (1) To secure a proper width and direction of streets; (2) The sound construction of buildings; (3) The diminution of danger arising from fire; and (4) The securing of more light, air and space round buildings.

The author then summarised the existing Acts and made detailed suggestions for their improvement.

The Paper then goes on to deal with the reinforced concrete regulations, and this part we give as follows:—

REINFORCED CONCRETE REGULATIONS.

The Regulations of the London County Council with regard to reinforced concrete are divided into ten parts, as follows:—

PART I. (General) describes the meaning of the term "reinforced concrete," specifies the application of the regulations and the nature of the materials to be used generally in reinforced concrete buildings, prohibits the use of any reinforcing metal for conducting electrical currents, and provides that copies of the plans and calculations, together with particulars of the materials to be used, are to be deposited with the district surveyor at the time when notice is served upon him under Part XIII. of the 1894 Act.

PART II. consists of data to be used for the purposes of the Regulations. It contains a list of superimposed loads to be allowed for on floors to be used for various purposes and roofs of different slopes. It requires provision to be made for wind pressure, and specifies the maximum allowable ratio of span to the depth of a beam or cantilever. It prescribes the bending moments to be allowed for in beams and slabs with various degrees of end fixing under different dispositions of loading. It specifies the maximum stresses to be allowed on concrete of various proportions, and in steel. It makes provision for anchoring the ends of all tensile and shear reinforcement, and specifies the modular ratio to be assumed in beams and pillars.

PART III. (Beams and Slabs) contains regulations as to the minimum diameter and disposition of the steel reinforcement and the minimum thickness of slabs, and regulations as to notation and formulæ to be used in calculating the resistance moment in beams and slabs.

PART IV. relates to pillars and struts, and defines the terms "pillar" and "strut" and contains regulations as to the minimum and maximum diameter of reinforcing rods and their disposition and minimum area, regulations as to notation and formulæ and tables to be used in calculating the permissible load with various degrees of end fixing.

PART V. contains regulations as to walls, specifies the minimum thicknesses allowable, and the maximum area of openings permitted. It is to be noted that party walls and division walls are allowed to be constructed of reinforced concrete, and need not be of the thickness prescribed by the 1894 Act.

This is a variation from the provisions of the 1909 Act in the case of steel-framed buildings, which require all party walls to be of the thickness prescribed by the 1894 Act.

All brickwork, stonework, and plain concrete are required to be executed in Portland cement and mortar, and the allowable pressures on brickwork so built are specified. No regulations are given as to the allowable pressure on stonework built in cement mortar.

PART VI. contains regulations as to the allowable pressure on various beds of natural ground, and also limits the pressure on plain concrete in foundations.

These are similar to the provisions of Section 22 (24) and (25) of the 1909 Act in the case of steel-framed buildings.

PART VII. (Protection) contains regulations as to the minimum cover of concrete over the metal reinforcement.

PART VIII. (Materials and Testing). This part contains regulations as to the quality, size, and proportions of the materials to be used for making concrete, with a table of the ultimate compressive stresses which must be attained for various proportions at the end of one month and four months after mixing. Regulations are also made as to the manner of mixing and placing the concrete. Nos. 165 to 167 give directions as to the quality and treatment of the metal reinforcement.

No. 168 is printed amongst the regulations as to "steel," though it would apparently come better under the next set of regulations headed "Tests and Testing," which specify the maximum test loads to be applied; such tests are not to be made within ninety days of the date of laying the concrete.

PART IX. contains regulations as to "Formwork or Centering."

PART X. relates principally to workmanship, providing that the work shall be carried on as continuously as possible, that it shall be protected from too rapid drying and from frost, and that concrete shall not be laid when the temperature is less than 7 deg. above freezing.

No. 185 is a most essential regulation prohibiting the cutting away of concrete for pipes or any other purpose in such a manner as would reduce the strength of any part of the structure below the standard set up by the Regulations.

The remaining three regulations deal with woodwork, etc., fixed in or on the concrete.

There does not appear to be the same difficulty in determining whether the Regulations as to reinforced concrete will apply to a particular building as there is in determining whether a building is or is not a steel-framed building. Regulation 2 states that "These regulations shall apply *only* to the construction of buildings of reinforced concrete in which the loads and stresses are transmitted through each storey to the foundations by a skeleton framework of reinforced concrete, or partly by a skeleton framework of reinforced concrete and partly by a party wall or party walls."

Section 22 (29) of the 1909 Act provides that "It shall be lawful to make any addition to, or alteration of, or to do any other work to, in, or upon a building in accordance with the provisions of that section, provided that the loads and stresses in the part of the building so added or altered, etc. . . . are transmitted from the roof to the foundations by a skeleton framework of metal . . . and the provisions of this section shall in all respects apply to such part of a building as if the same were a separate building."

There appears to be no similar provision in the regulations, though from the wording of Clause 6 it appears to be contemplated that additions or alterations may be carried out under the Regulations. In my opinion it would have been well to have inserted a similar clause to Section 22 (29). It would also have been better to make the Regulations applicable in all cases where reinforced concrete is used. In the present circumstances there would appear to be no regulation as to reinforced concrete floors in ordinary or steel-frame buildings.

In the Regulations as originally drafted there was a provision similar to that in Section 22 (18) of the 1909 Act—viz., "In every building of the warehouse class a notice shall be permanently exhibited in a conspicuous place on each storey of such building stating the maximum superimposed load per square foot which may be carried on any part of the floor of such storey." This provision does not appear in the latest revision of the regulations. In my opinion such a regulation is even more desirable in the case of reinforced concrete than in the case of steel framing, because in the former it is impossible to ascertain by inspection the strength of a floor.

No allowance is made for high-tension steel, so that apparently those systems which use mesh and other high-tension steel reinforcement would be inadmissible in buildings erected under these regulations. In the R.I.B.A. Second Report such steel is recognised, with a provision that the stress on it shall not exceed one-half the stress at the yield-point of the steel, and a maximum of 20,000 lb. per sq. in.

It is also desirable to make a provision both under the Regulations as to reinforced concrete and in the Act as to steel-frame buildings that the drawings and calculations should be deposited with the district surveyor some time (at least one month) *before* the work is to be commenced. In the present circumstances, as notice is only necessary two days before the work is commenced, sufficient time is not given to the district surveyor to go through the scheme and check the loads which will come on the foundations before the foreman is clamouring at the door wanting the district surveyor to pass the bottoms for the stanchions and pillars.

The regulations as to testing do not specify who is to require tests of the completed structure. Testing may be found necessary "by reason of any sign of weakness or faulty construction appearing." But the contractors' idea of "signs of faulty construction" may be very different from the district surveyor's. Who, then, is to decide? The New York regulations provide that "The contractor may be required to make load tests on any portion of a reinforced concrete structure within a reasonable time after execution." It is also noticeable that the New York regulations require the construction "to sustain safely a load of *twice* the superimposed load for which it was designed," whereas the London County Council Regulations limit the test load to $1\frac{1}{2}$ times the superimposed load for which the construction was designed.

The London County Council regulations as to protection are considerably less stringent than the New York regulations. The latter require a minimum cover of 2 in. of concrete in columns and girders, $1\frac{1}{2}$ in. in walls and beams, and 1 in. in floor slabs; the former only require $1\frac{1}{2}$ in. in pillars, 1 in. in beams, and $\frac{1}{2}$ in. in slabs.

The allowable stresses in concrete of 1 : 2 : 4 mixture under London County Council Regulations and New York regulations are compared below :—

	L.C.C.	NEW YORK.
	LBS. PER SQ. IN.	
Extreme fibre stress on concrete in compression	600	650
Concrete in direct compression	600	500
Shearing stress	60	40
Adhesion stress when bars hooked at ends	100	80
Adhesion stress, bars otherwise effectively anchored	60	80

CONCLUSION.

When I was asked to give a short paper on "the whole subject of the London Building Acts, including the Steel-frame Act and the Reinforced Concrete Regulations," I felt I was asked to do an impossibility. My paper is very far from dealing exhaustively with the subject, and I am fully alive to its shortcomings. I can only, however, submit it for what it is worth, and trust that the discussion to follow will bring out many useful suggestions for future amendments.

DISCUSSION.

The President (Professor Henry Adams, M.Inst.C.E., etc.) said that Building By-laws were a perennial source of discussion. There were perhaps no Regulations more difficult to make and none that gave so little satisfaction to those concerned in obeying them. From time to time the objectors fell into line, and modifications ensued. The time now appeared to be ripe for another general consolidation of the various Acts and Regulations, and this was a good opportunity to ventilate grievances.

With regard to the footings for brick walls, under the London Building Act, it was worthy of note that in the North of England it was customary to build without footings, although in the London Building Act they were told they were essential. The District Surveyors' fees on repairs sometimes exceeded the cost of the repairs themselves. In his own practice he made eight times the thickness the limit of projection—that was to say, a half-inch plate might project not more than four inches. That was the extreme, and the same rule applied to girders. Where there were two or more plates he should consider "properly riveted" meant "tack rivets," not more than six inches pitch. He agreed with the Author as to the riveting in the base plates, but it was seldom that the base was machine planed before it was connected to the base plate, and that was why the whole stress must be capable of being taken by the rivet. With regard to riveting through a thickness of four or five inches, it was not so many years ago that there was no machine riveting, and good work was quite possible by hand through this thickness. All that was necessary was that the point should be cooled before the rivet was driven, so as to cause it to swell in the centre under the blows of the riveters. A notice should be permanently exhibited on every floor of a reinforced concrete warehouse, stating the maximum superimposed load for which the floor was designed. The Council of the Concrete Institute pressed for this Regulation, but the London County Council had no power to make it.

Mr. H. D. Searles-Wood, F.R.I.B.A.: One of the great things they could do in a new Building Act was to avoid the terrible delay which occurred in getting approvals from the London County Council. The thing was crystallised to such an extent that it ought not to be necessary to spend all the time they did in getting their plans passed. Surely it would be possible now to have a little more scientific treatment of the subject and not so many schedules. Let the designer show that he had properly calculated his structure, and then give him a free hand.

Mr. W. G. Perkins (District Surveyor for Holborn) observed that the present Acts undoubtedly required to be amended and modified in order that they might be more easily understood by persons who were building in London, so as to admit of more recent improved and scientific methods of building being adopted freely and without the necessity of going to the London County Council for special sanction in individual cases. He was not sure that it was a good thing to give the district surveyors more discretion in the various cases that arose under the Act. Sometimes a discretion was a two-edged sword. He complained that the City was excluded from the law governing the general line of buildings. He did not agree with Mr. Hills that the district surveyor had unlimited discretion as to the interpretation of Section 78 of the 1894 Act. His opinion was that the district surveyor must see that the whole of the rules of the Act were complied with in the case of a public building, but he had power to ask for anything further that he thought necessary in the way of construction.

Mr. E. Flander Etchells, A.M.Inst.C.E., etc.: The clauses of any amended Act ought to be drafted by architects and engineers to save them from the lawyer-drafted clauses which enabled them, under Section 8 of the Act of 1894, to lay out a street, provided they said they were doing something else.

The publications of the Engineers' Standards Committee were frequently referred to in Acts of Parliament, and it might be an advantage to adopt a similar suggestion instead of incorporating schedules in the new Building Act.

He admitted that there was a desire on the part of all of them to remove anomalies from the present Building Act, but he cautioned them that anomalies could not be done away with altogether. They could only hope that the anomalies to come would not be so confusing and distracting as some of those to which their legal friends had already subjected them. With regard to Building Laws in general, the idea of Building Law was that which was most in accordance with Natural Law, and in that he included Mechanics, Hygiene and Ethics, and also the true laws of sound finance in the highest acceptance of that term.

He reminded the meeting that 7,000 dangerous structures were dealt with every year, and only one or two got into the papers. Many of the criticisms should not be addressed to the building authority, because they had endeavoured to find a compromise between extreme views; they should be addressed to other engineers who held these extreme views. "The greatest common measure of fads" was brilliant as a phrase, but it did not compare with the actual facts.

Mr. Wm. Woodward, F.R.I.B.A., remarked that Mr. Hills took a reasonable view of the Building Acts. It was possible, however, for a District Surveyor to take such a hard and fast line in the administration of the Acts as really at times to make them very oppressive. He was old enough to remember the preliminary discussions to the Act of 1894. At that time those who were in authority in the London County Council desired to frame some of the sections in a confiscatory manner so as to have a dig at the ground landlords. The Building Act of 1894 did sometimes interfere with the value of property, but, speaking personally, on the whole he thought it protected the public from many of the drawbacks and nuisances in existence before it was passed. They all hoped for a new Act which would contain in itself, without reference to other Acts, all that was necessary for carrying on the work of building in London. There was great difference of opinion amongst District Surveyors as to party walls, but his contention was that such a wall became an external wall beyond the height where it separated buildings.

Mr. Edward Dru Drury, F.R.I.B.A. (District Surveyor for St. Margaret's, Westminster), reminded the meeting that, in a case of his against the Army and Navy Stores some years ago, the Court of King's Bench decided that a party wall ceased to be a party wall as soon as it reached 15 in. above the lowest building. When they had a new Act it would be important to have in it a definition of a division wall.

Mr. C. S. Melk, M.Inst.C.E., referring to the manufacture of reinforced concrete, said it was useless to make very stringent provisions for the design of work unless some strict supervision was exercised over the carrying out of the work. The Local Government Board in their Regulations, it appeared to him, had done a great deal to discourage the introduction of reinforced concrete, and he hoped to live to see these Regulations and the County Council Regulations as to reinforced concrete very much improved and condensed.

Mr. J. Ernest Franck, A.R.I.B.A., disagreed with previous speakers, and thought the drafting of the new Act should be left to lawyers, who would bear in mind the continuity of the Act with the preceding Acts. The Act of 1894 put a premium on bad building. The new Act should be framed on broad principles and certain ratios should be given, which should be worked to. Under the 1894 Act many brick walls were erected which should not be allowed, while in other cases they had to build a wall which was unnecessarily thick.

Mr. Lawton R. Ford, A.R.I.B.A. (District Surveyor for St. James's, Westminster) : A consolidated Act, with amendments doing away with discrepancies and various anomalies was one of the pressing needs of London, as great changes had taken place in building construction during the past twenty-one years.

Mr. Percy Hunter, A.R.I.B.A. (District Surveyor for South Lambeth), maintained that, however much they consolidated their present Acts, there would be advances in building construction, and as far as he could make out, whatever Act they passed in any given year, ten years after they would want an Amending Act. If the principle of responsibility for his design were taken away from the architect they might as well abolish him altogether. One of the principles he laid down with regard to reinforced concrete construction was that there should be special supervision of the work.

Mr. S. Bylander thought the responsibility should be shared between the designer and the builder. The law should be so amended that the engineer should accept responsibility for design. In order to simplify the design of a building next to another building, it would be advisable, where reinforced concrete was used, that plans showing the work as executed should be deposited with the London County Council, and architects should have the opportunity of examining these by paying a small fee, in the same way as company prospectuses were available at Somerset House.

Mr. Allan Graham, A.R.I.B.A., expressed the opinion that the fruit of the paper would be not greater stringency, but greater leniency in connection with the Act. Everyone who attempted to alter or deal with the London Building Act ought to have an architectural training and understand the tremendous difficulties the architect had to face in complying with the regulations as regarded open space in a city like London, where ground was so expensive and where the regulations tied him up. He would much prefer to have the London Building Act, 1894, remain as it was than accept some of the suggestions which had been put forward. The whole tendency of regulations was to get worse and worse. Everyone had his fad, and they got the greatest common measure of fads as the result of all the deliberations incorporated in the by-laws. If there were one authority in London, as in Glasgow, an architect would come into his own and not be harassed by all these various troubles which hindered him in his profession. In Glasgow all charges were borne by the Corporation; a client had not to pay for the passing of his plans at all.

Mr. Percy H. Simco dealt with that portion of the Act relating to steel-frame buildings. This had been in many respects a useful Act, inasmuch as it had stopped by law all "scamping" in steelwork in public buildings. But on the other hand, to those who had had to administer it and work under it, it had been somewhat a curse inasmuch as there was so much ambiguous wording in it. The building owner so far had had all the benefit. It would be as well that the architects of London should get their clients to make up their minds definitely before the plans were actually passed and the work put in hand. Clauses bearing on the actual engineering and constructional points ought to be drafted by those whose business it was to administer and work under the Acts, and then they might get some plain and straightforward wording. The Act should be on broad lines, and the district surveyors should be able in many instances to use their own judgment, because points were bound to crop up in construction which could not be covered by a cut-and-dried Act. In the event of any disagreement with the district surveyor, a tribunal of appeal should be set up, which should be easy of approach, and so delays could be avoided.

Mr. W. Cyril Cocking thanked the district surveyors for the help they gave to engineers and architects in the design of public buildings. The value of a job had, to his knowledge, been increased 25 per cent. because of the kindness of the district surveyor in showing the architect how to do his work and how to get the best value out of his design. That was a work for which they got no remuneration whatever, and very little thanks. Architects should be compelled to notify the district surveyor of every alteration in the structure. It should be made part of the amendment of the London Building Act that all calculations, plans and details for uncompleted structures must be filed with the copy of the lease, so that when the job was completed the information would be at hand. All steelwork designed under the 1894 or the 1909 Act should be submitted to the district surveyor for his approval in the interests of public safety. To him it was a pitiful sight to see a district surveyor poring over calculations. The district surveyor was much more wanted on the job to find out things that the engineer and the builder left out. Notices were not of much value; what was wanted was a periodical and surreptitious visit of the district surveyor.

Mr. Ewart S. Andrews, B.Sc., considered that any rules made should not specify the projection beyond the beam, but the projection beyond the centre of the rivet. One of the chief difficulties arose from the allowable stresses being specified in the Act, apparently intentionally, to drive them to deal with their stanchion as one with fixed ends. There was, he believed, no scientific justification whatever for the hinged end being reduced to the large extent it was on comparatively short lengths. The several stresses approached to a common value in a very short column, and the effect of the great reduction in the allowable stress on the hinged ends under the Act was to drive people to have what they called fixed ends. If, on the other hand, the figures were made more lenient and more in accordance with fact, he believed the difficulty would be solved to a very large extent. They should have some indication as to when an end was to be considered fixed, but for that they would have to wait some time. The number of rivets connecting the base plate to the main column really had to do with the question of fixity. If an end were perfectly fixed it would bear a very large bending moment before buckling would take place. As to the load on floors, he agreed that 100 was rather a high figure. The difficulty would be met if the rule specified not only a super load, but also a concentrated load.

Mr. H. Kempton Dyson, Secretary, Concrete Institute, said: Ten years ago, in a leader in the *Builders' Journal* for April 5th, 1905, I wrote: "It is important that the present Acts should be revised, and we consider the Council will be well advised not to wait longer than next year. We hope no half measures will be attempted, but that the old Acts will be repealed and a comprehensive new Act brought in, because the present Acts and amendments are quite confusing enough to professional men; a building ordinance should be businesslike in form."

Hitherto the suggestions of professional bodies have been treated with scant courtesy. After receiving suggestions the Building Act Committee should send their altered clauses for further suggestions, and not try to rush through a Bill in defiance of all interested bodies."

Now the procedure adopted with the form given to the proposed Reinforced Concrete Regulations conform most closely to my old ideals. The co-operation of the professional societies has been sought and their recommendations treated with courtesy, the altered proposals have been sent again and again for further suggestions, while the fact that they are to be Regulations and not an Act of Parliament will permit of their being revised and kept up-to-date. The language is straightforward and more intelligible than of old, but I still await that desired complete overhauling of the existing Building Acts. The changes within recent years in the outlook on methods of constructing buildings point clearly to the advisability of having the Building Ordinance in a form which can be easily modified from time to time, and it seems to me that Regulations are the elastic form required. We need a Code Napoleon for our Building Law, and if all legal precedent for this country is against it, then let such matters of general policy as the width of streets, lines of frontage and rights of adjoining owners, space about the rear of buildings, be enacted, but as regards the details of construction let them be given in Regulations that can be modified from time to time. The present Acts are not only very disorderly and involved in expression but are most unscientific. The Steel Frame Act and the Reinforced Concrete Regulations have specified stresses, methods of calculation and workmanship, and it seems to me that the same ought to be done for the other materials and constructional methods in vogue (such as timberwork, stonework, brickwork), and some which may yet be in vogue (such as reinforced brickwork, reinforced stonework, reinforced steel, reinforced glass, reinforced papier maché, nickel-steelwork, aluminium-alloy work, concrete slabs and blocks for hollow and solid walls, and so on). The regulation of the thickness of walls by the height of the storey is out of date in this scientific age; the thickness of walls should be determined by the work required thereof as to stability, resistance to fire, and hygienic qualities such as damp-proofness.

The health of the occupants of buildings is not thoroughly studied at present, for the proper number of changes of air per hour are not regulated, while unnecessary restrictions are placed upon designers who wish to give the maximum amount of light. Among other reforms, I should like to see greater facilities given for enabling economy to be effected in the construction of foundations of buildings. The maximum load that the ground will safely sustain should be put upon it, and footings could often be eliminated. It is important to know the character of the subsoil in the preliminary stages of the design of a building, and there is probably a mass of information on the subject in the possession of the London County Council's various departments and of the district surveyors of London which ought to be codified and made easy of reference by the public, who might well be asked to pay for the work of codification, for it would save the expenditure of many thousands of pounds a year on building operations.

MR. HILLS' REPLY.

Mr. Hills, in reply, said that what they really wanted was an Act embracing all the law in regard to buildings, and they might have some schedules or regulations added to it. A man who had to draw up a Building Act must be accustomed to that sort of work. Surveyors would make as much "hash" of it, if they tried to do it without a lawyer, as if a lawyer tried to do it without a surveyor. The suggestion that the new Building Act should apply universally did not find approval in his eyes. He did not think that the rules which applied to the suburbs could well be brought into the City. It was quite reasonable that an ancient city like the City of London should be exempt from all the rules which were applicable to more modern parts. Space was very valuable in the City, and it was not necessary to have the same amount of open space for an office as for a dwelling-house. The construction of public buildings was left to the district surveyors. In all other points public buildings must comply with the requirements common to all buildings. He agreed that there ought to be a stringent supervision of reinforced concrete work, and, in one case, that had resulted in the detection of the omission of the steel. If they left out 14 15ths of the whole strength of the thing, they would see that supervision was necessary.

It had been suggested on the one side that district surveyors should have a great deal more discretion, and on the other hand that that would be a dangerous thing. Being a district surveyor, he might not be unbiassed, and therefore he proposed to leave it where it was, with a considerable divergence of opinion. In conclusion, he pointed out that no new Act could possibly be passed during the continuance of this terrible war, but they could certainly prepare for it.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

ORNAMENTAL REINFORCED CONCRETE BRIDGE, CLACTON-ON-SEA.

The development of Clacton-on-Sea affords an admirable illustration of what a zealous and energetic local authority can do towards the making of a popular health and holiday resort, and during the past two years the Council has everywhere impressed itself to the great advantage of the town, whether regarded from the point of view of the resident or from that of the holiday-maker.

A magnificent band and entertainment pavilion has been constructed on the sea front at a cost of £14,000, which work was designed and carried out by administration by the Surveyor to the Council, Mr. D. J. Bowe. Adjoining the site of this pavilion was a roadway running down from the centre of the town to the pier, and on each side of this roadway was a row of unsightly small shops which used to obtrude themselves. These shops were purchased by the Council from the Pier Company at a cost of £5,000, the site was entirely cleared, and on their former position artistic rock gardens have been constructed and laid out in a very pleasing manner.

To connect the East and West Cliffs and so make a continuous promenade of over two miles along the top terrace of the sea front, as well as to give quicker access to the pavilion, the bridge, here illustrated, was con-

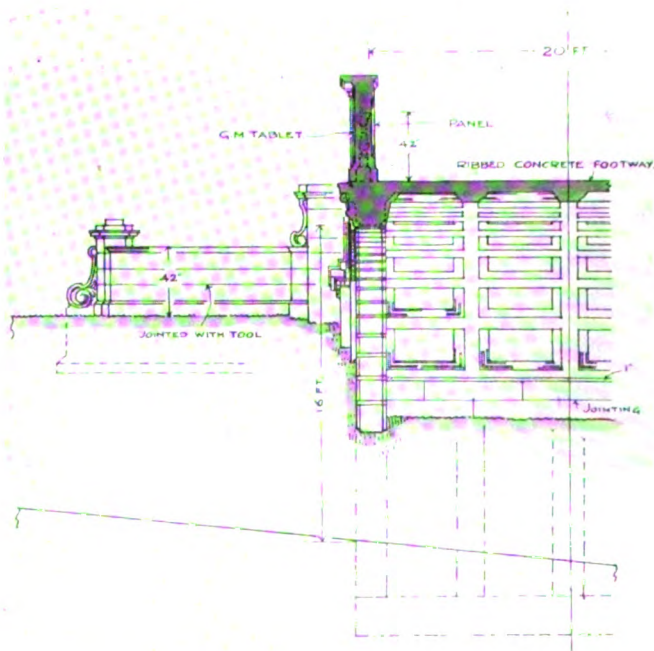


Fig. 1. Half Section through Centre of Bridge.
ORNAMENTAL REINFORCED CONCRETE BRIDGE, CLACTON-ON-SEA.

structed last year. It is of reinforced concrete on the "Piketty" system, and is in complete harmony with the Band Colonnade, being in the Renaissance style and purely Italian in feeling. It is probably one of the best examples of architectural treatment being incorporated with reinforced concrete work. The voussoirs are surmounted by a dentil course and balustrading, with a central pediment on each side, carrying on the outside of each the town's coat of arms cast in bronze.

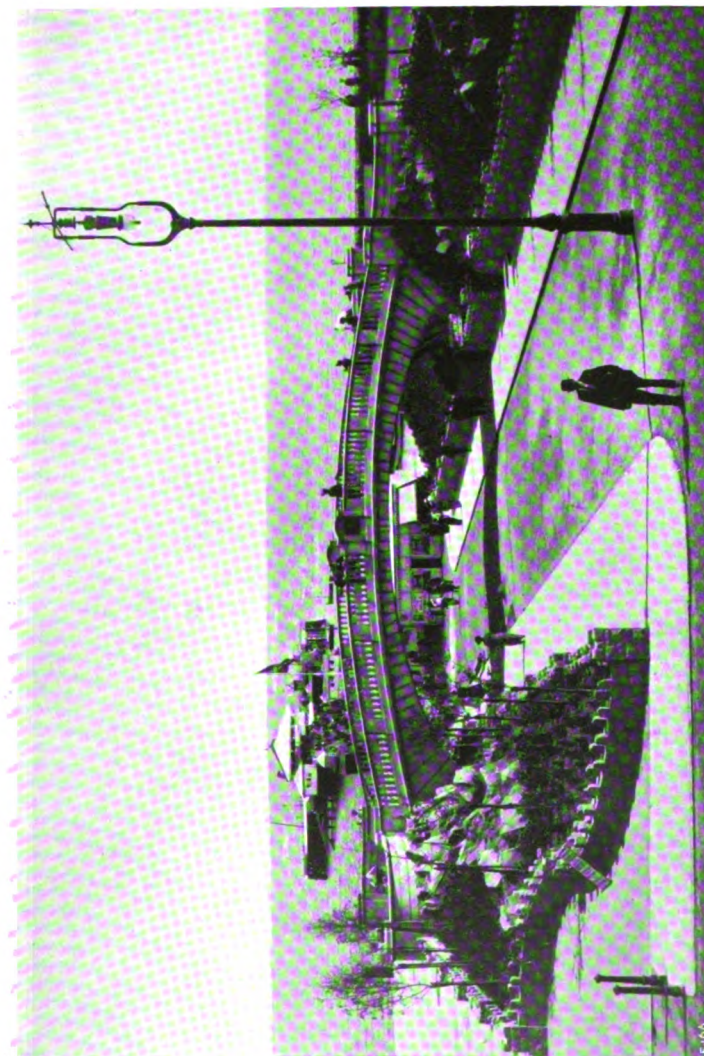


Fig. 2. View of Finished Bridge.
ORNAMENTAL REINFORCED CONCRETE BRIDGE, CLACTON-ON-SEA.

The clear span between the abutments is 60 ft. (the roadway beneath being 45 ft. wide), the width between the parapets being 20 ft., whilst the headway above pier gap is 15 ft. This latter height allows the lifeboat being taken on and off the pier. Inasmuch as the bridge is for pedestrians only and cannot, from its position, be used for any vehicular traffic, it was designed to carry a super load of $1\frac{1}{2}$ cwt per sq. ft. The maximum stress on foundation is 35 lb. to the sq. in., the bearing capacity of the foundation level being estimated at 3 tons per sq. ft.

The foundation slabs are 14 ft. by 22 ft. by 7 in. in thickness, and the design of the

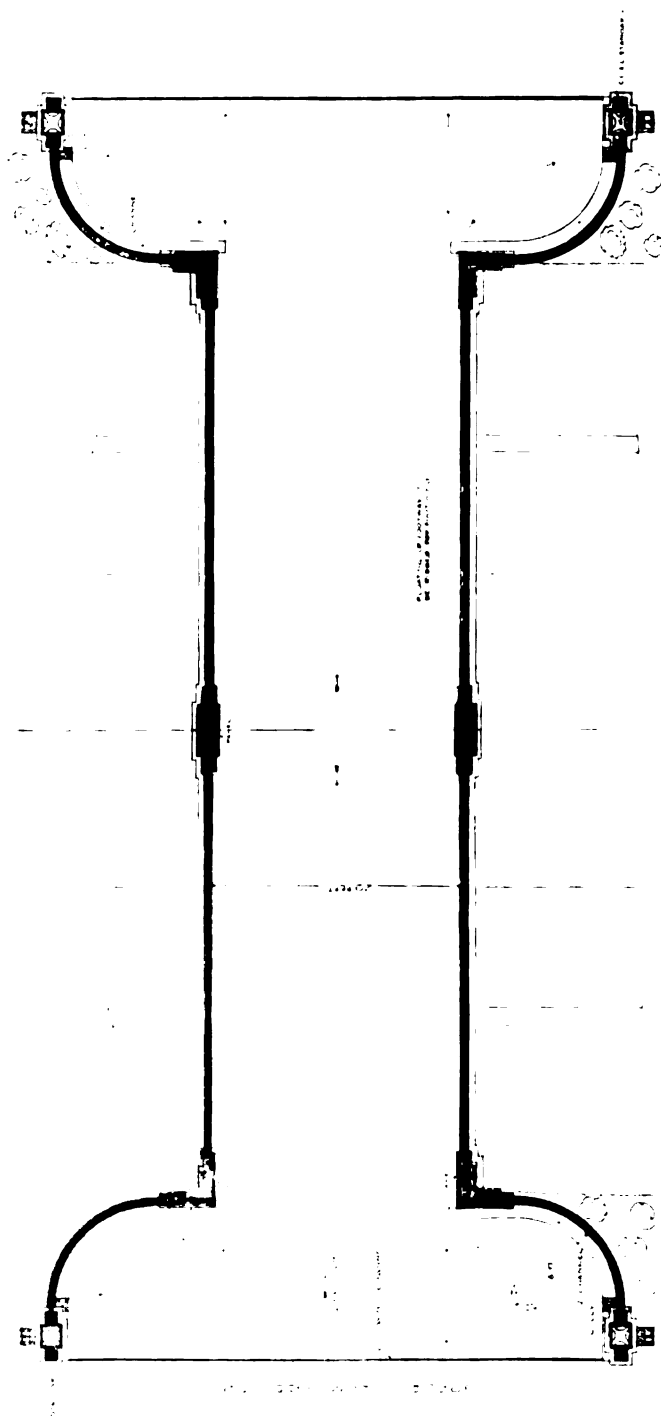


Fig. 3. Plan.
ORNAMENTAL REINFORCED CONCRETE BRIDGE, CLACTON-ON-SEA.

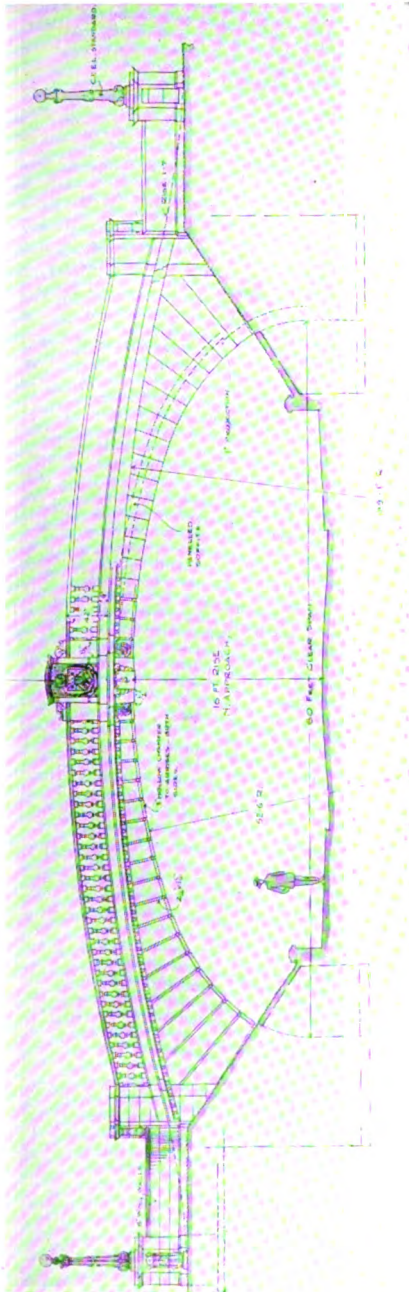


Fig. 4. Elevation from North Approach.
ORNAMENTAL REINFORCED CONCRETE BRIDGE, CLACTON-ON-SEA.

bridge is on the cantilever system. The outside arch ribs are $20\frac{1}{2}$ by 9 in., and the centre one $20\frac{1}{2}$ by 12 in., whilst the ribs are connected transversely by beams varying from 5 ft. $2\frac{1}{2}$ in. to 5 ft. 9 in. apart, and from $6\frac{1}{2}$ in. by 10 in. to $6\frac{1}{2}$ in. by 12 in. in size. The decking slab is $3\frac{1}{2}$ in. in thickness and the underneath of the bridge is beautifully panelled. The wing walls are 7 in. in thickness, and the centering of the arch was not struck until the filling above the foundation slabs was completed up to a height of 9 ft.

The floor of the bridge is of La Brea Asphalte, 1 in. in thickness, and the concrete was to the following proportions, viz.: cement, 510 lb.; sand, 10.75 cu. ft.; stone, 21.50 cu. ft.

Three months after the completion of the work the bridge was tested by an evenly distributed load of 110 tons being placed thereon and remaining there for some two hours before being gradually removed.

The specification stipulated that the deflection under the test should not exceed $1/600$ th of the span, viz.: 1.3 in., and it was practically unmeasurable.

The design and architectural treatment of the whole of the scheme was the work of Mr. Daniel J. Bowe, the Council's Surveyor, whilst the reinforced concrete details were supplied by Messrs. Paul Piketty and Co. The work in connection with the bridge was carried out by Messrs. W. Archer and Son, of Gravesend, under Mr. Bowe's supervision and control, with Mr. W. T. Morrison as clerk of the works. The cost of the bridge, with coat of arms panels, wrought iron electric light standards, and a few other miscellaneous items, was just over £1,000.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

Reinforced Concrete in Practice. By A. Alban H. Scott.

London: Scott, Greenwood and Son, 8 Broadway, Ludgate, E.C. 178 pp. + viii. Price 3/- net.

Contents.—General Notes—Materials—Testing of Materials—Centering—Preparation of Steelwork—Concrete—Striking of Centering—Cutting Away and Making Good, Surface Treatment and Finish—Work Requiring Special Method—Fixing of Machinery, Plant, etc.—Testing of Finished Structure—Contraction, Expansion.

The author states a great truth in his preface when he says that there is no building material so lasting and requiring less maintenance than reinforced concrete, but it should only be employed when the work is under constant, careful, and experienced supervision, and when every material in the work is tested; and this volume is very welcome because it puts forward information useful to those engaged in reinforced concrete work, which information is based on the right principles and consequently it deserves to be popular. The notes are presented in such a manner that the interests of all parties are considered, and while those points conducive to good practice are impressed upon the reader, many useful hints dealing with the economical execution of the work are given for the benefit of the contractor.

In the general notes the author states that great improvements have been found in work executed where the foreman explains to the ganger and he in turn explains to his men in their own particular language the different functions the materials fulfil. The preparation of the drawings and the lay-out of the work on the site are also discussed. After dealing with the materials some very good practical notes on the centering are given, and these are illustrated with numerous photographs of actual work, as is also the following chapter on the preparation of the steelwork.

The question of concreting is fully considered from the point of view of obtaining good work, and some useful notes are given in connection with the temporary suspension of concrete, which is an important problem. The work described in the chapter dealing with that requiring special method includes chimneys and other constructions subjected to high temperatures, tanks, and piles.

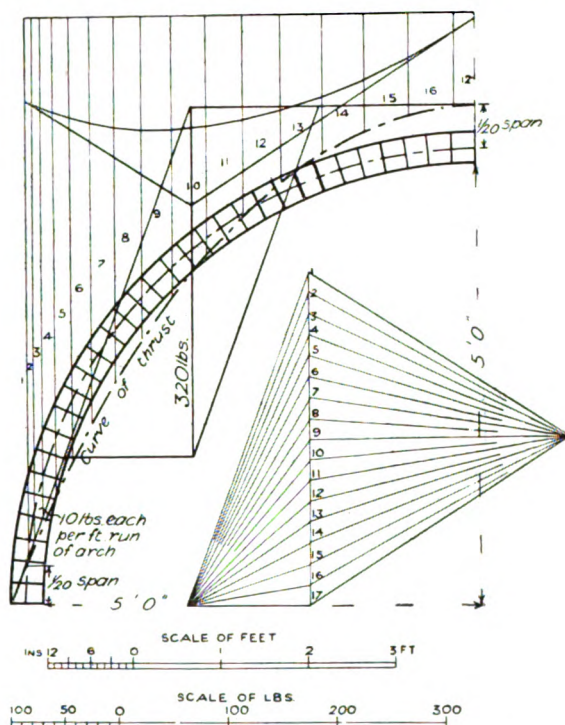
The book is well written and well illustrated throughout, and it is practical in character, rendering it suitable for a great number of readers of all classes, and it can be recommended as a handy volume which will prove very useful to those engaged in the design, supervision, and execution of reinforced concrete work.

CORRESPONDENCE.

Under this heading we invite correspondence.

To the Editor of CONCRETE AND CONSTRUCTIONAL ENGINEERING.

SIR,—The solution of the arch problem in your last issue, p. 230, is interesting, but is not applicable to the case of the brick arch in question. This was a half-brick arch, built as a cover only, which fell directly the centering was removed. My solution of the case is given in the diagram herewith, where the curve of thrust is assumed to pass through a point one-twentieth of the span above the neutral axis at the crown, and



through the neutral axis at one-twentieth of the span above the springing. This is an empirical solution only, and Mr. Andrews was asked if he could prove it by mathematical analysis.

The horizontal thrust is found graphically to be about 118 lb., and the bending moment $118 \times \frac{1}{20} (10 \times 12 + 4.5) = 734.55$ lb.-in. The maximum stress at the crown will then be $\frac{W}{A} \pm \frac{M}{Z} = \frac{118}{12 \times 4.5} \pm \frac{734.55}{\frac{1}{8} \times 12 \times 4.5^2} = 2.185 \pm 18.137 = 20.3$ lbs. per sq. in. compression and 15.95 lb. per sq. in. tension, so that, had the arch been built in cement mortar and been allowed to set before the centering was removed, it would probably have just succeeded in standing.

HENRY ADAMS.

INDUSTRIAL NOTES.

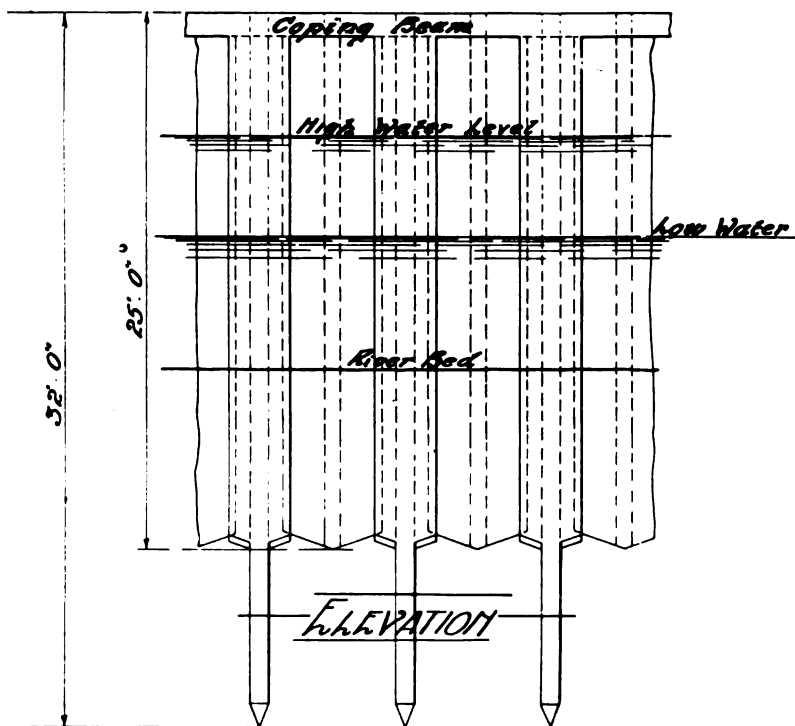
These pages have been reserved for the presentation of articles and notes on proprietary materials or systems of construction put forward by firms interested in their application. With the advent of methods of construction requiring considerable skill in design and supervision, many firms nowadays command the services of specialists whose views merit most careful attention. In these columns such views will often be presented in favour of different specialties. They must be read as ex parte statements—with which this journal is in no way associated, either for or against—but we would commend them to our readers as arguments by parties who are as a rule thoroughly conversant with the particular industry with which they are associated.—ED.

A NEW METHOD OF REINFORCED CONCRETE SHEET PILING.

Reinforced Concrete Wharf at Great Yarmouth.

A NEW method of reinforced concrete sheet piling has recently been employed for the construction of a new wharf front at Great Yarmouth on the River Yare. The work was carried out for Messrs. H. Newhouse and Co., A.B.C. Wharf, under the supervision of Mr. F. G. Turner, Engineer to the Harbour Commissioners.

The new method of piling employed, which goes under the name of "The Coignet-



Ravier System," consists in driving at intervals of about 6 or 7 ft. a certain number of piles fitted with lateral wings or sheeting, the intervening space between the sheeting and the main piles being filled up by other similar wing piles driven at the back of the front row, the object being to form a continual quay front by means of these wing

piles, which are held in position by being driven sufficiently into the ground by means of a steam monkey, and held back by means of reinforced concrete land ties and anchor plates.



VIEW OF QUAY WITH TIMBER FENDERS.



KING PILES MATURING UNTIL READY FOR DRIVING.

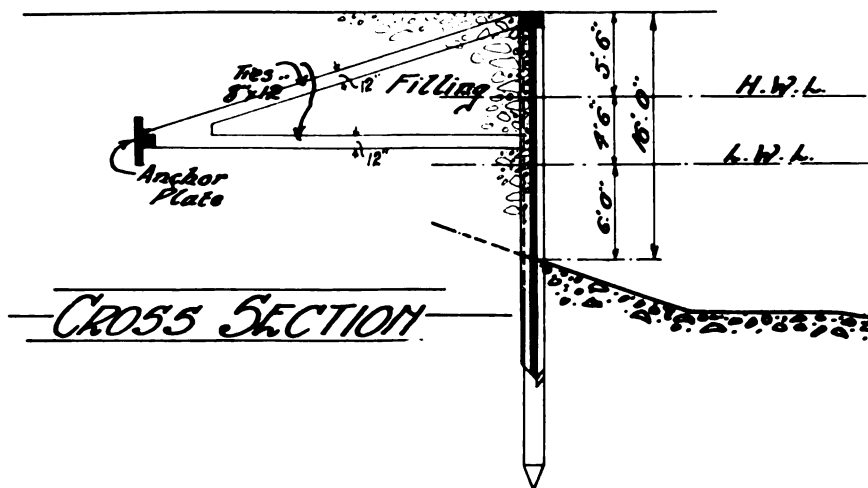
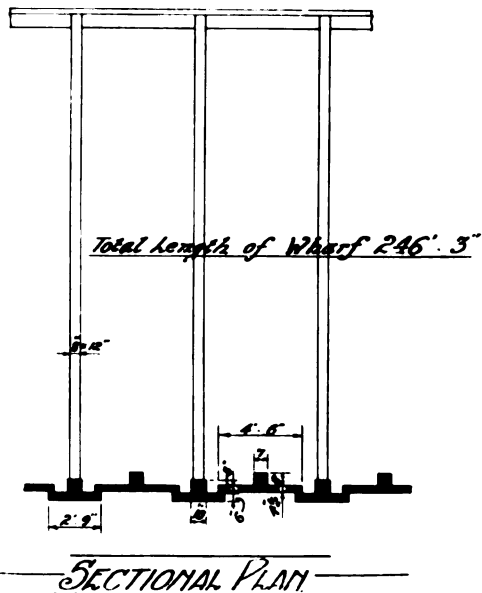
The principal advantages claimed for this method are that a much smaller number of piles requires to be driven than in the case of continuous king piles and sheet piles,

and also a certain amount of economy in the actual steel and concrete required.

The plans were prepared by Messrs. Edmond Coignet, Ltd., of 20, Victoria Street, Westminster, who are the patentees of this new method, which has already been employed recently for the construction of a wharf front for the Harwich Gas Works. This new method of sheet piling has also been extensively used on the Continent.

The photographs reproduced here show the general appearance of the finished structure, with timber fenders for the protection of the reinforced concrete work. The total length of the quay front will ultimately be about 246 ft. The total length of the main piles from the top to the end of the shoe is 32 ft., and the length of the wings measures 25 ft. with a width of 2 ft. 9 in. The intermediate wing piles are also 32 ft. long with a panel of 25 ft. in length by 4 ft. 6 in. in width.

The work was carried out by Mr. A. W. Chastney (Wick and Sons, Ltd.), of Norwich



POPULAR USES.

Under this heading it is proposed from time to time to present particulars of the more popular uses to which concrete and reinforced concrete can be put, as, for instance, in the construction of houses, cottages, on the farm and on the road. Previous articles will be found in our issues of December, 1912; January, March, July, October and November, 1914; and January and February of this year.—ED.

CONCRETE POSTS FOR LETTER BOXES IN U.S.A.

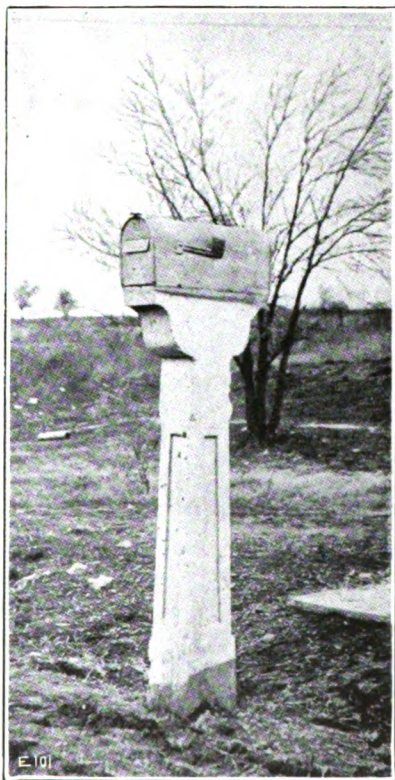
The accompanying illustrations show a novel use of concrete. It is a post supporting a letter box on a rural free delivery route near Dallas, Texas. The concrete features consist of the post or support for the box. Wooden posts as used there are always subject to rapid decay at the ground level, where alternate wetting and drying takes place, and to avoid the necessity of renewals and repairs the concrete post was made.

The form for casting a post of this kind is shown below. The post should be about 7 ft. long. Planted at a depth of 3 ft. this would leave the box at convenient height for the delivery or extraction of mail. The form, as shown on the drawing, is simply a three-sided box providing for a post 6 in. square. The form is placed on the ground in horizontal position, with open side up, and filled with well-tamped concrete to the depth of about 1 in. Then $\frac{3}{4}$ -in. reinforcing rods are placed at either side, leaving about 1 in. space between each rod and the side of the form. The box is then filled to within 1 in. from the top and two more reinforcing rods are placed on the concrete. The form is then filled to the top and the concrete struck off. The rods may go straight from the bottom to the top of the post, as it will not be necessary to curve them outward at the bracket.

The form or mould may be made with one or more braces nailed across the upper or open side to prevent the sides from spreading when the concrete is tamped. The bracket effect at the top is obtained by inserting extra pieces of wood at the corners, as shown. If preferable a post in the shape of a letter T may be made, eliminating the bracket feature. A perfectly plain post without projecting top would also answer the purpose.

To provide for fastening the letter box to the post, bore two or perhaps four holes through the board at the top of the form. Insert in these holes the bolts with the heads down, or inside the form. The bolts will be embedded in the concrete when it is placed, and when the form is removed the threaded ends of the bolts will project slightly above the top of the post. Holes cut in corresponding position in the bottom of the letter box will permit the bolts to pass through and the box will be secure when nuts are placed on the bolts.

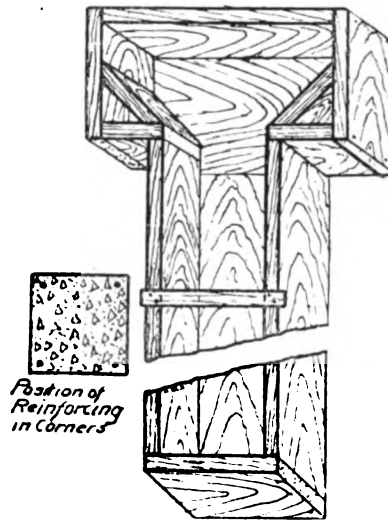
Mix the concrete in the proportion of 1 part Portland cement, 2 parts sharp, clean sand and 4 parts crushed stone, ranging from $\frac{3}{4}$ in. to 1 in. in size. Allow the concrete to remain in the forms for at least 24 hours. When the post is removed protect it from



CONCRETE MAIL BOX POST NEAR DALLAS, TEX.

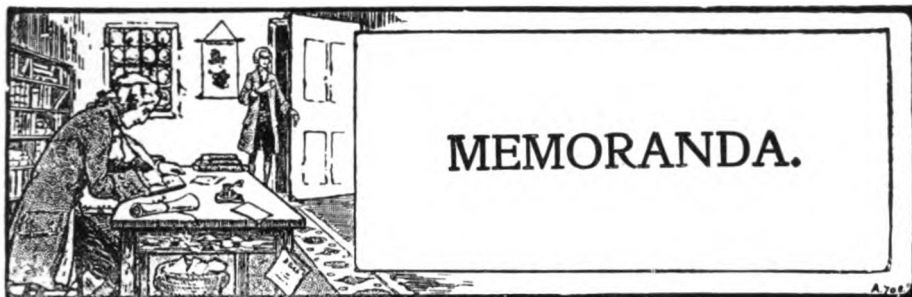
freezing or, if made in summer, from hot winds and sun. Wet it thoroughly for a week or ten days after removing it from the forms.

Concrete posts do not warp, decay or burn. When used for fencing they keep in better alignment than wooden posts. Concrete is now largely used in America for fence



FORM FOR CASTING A CONCRETE POST SIMILAR TO THAT SHOWN ABOVE.

posts, clothes poles, hitching posts and gate posts. Concrete fence posts have been made at an average cost of less than 25 cents each, notwithstanding the fact that all material was purchased, and even in well-timbered districts they are being substituted for wooden posts on account of their low first cost and everlasting qualities.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The National Road Conference and Exhibition (1915).—This Conference and Exhibition is to be held in London at the Royal Horticultural Hall, Vincent Square, Westminster, from June 25th to July 1st.

It will be opened on June 25th by the Right Hon. Herbert Samuel, M.P., late President of the Local Government Board. The Conference is organised by the County Councils Association, and will take place concurrently with the Annual Meeting of the Institution of Municipal and County Engineers.

The preliminary arrangements are as follows :—

June 25th and 26th.—Annual Meeting of the Institution of Municipal and County Engineers, when the following subjects will be discussed :—

1. The improvement and maintenance of highways in connection with modern traffic conditions.
2. The use of reinforced concrete in buildings, road construction, etc.
3. Town Planning.
4. Haulage by mechanical means *v.* horse power from an economical point of view.
5. Restriction and control of heavy traffic both as regards weight and speed.
6. The maintenance and strengthening of iron and steel bridges.

June 28th.—Conference on "The adequacy of bituminous roads, with special reference to the subject of corrugations," and other matters.

June 29th.—Conference on Heavy Traffic.

June 30th.—*Morning*: Meeting of Executive Council of County Councils Association. *Afternoon*: Conference on the Classification of Roads.

July 1st.—Conference on the Reconstruction of Roads in Belgium arranged jointly by the County Councils Association and a Special Committee which has been formed in connection with the Rebuilding of Belgium.

Arrangements as to other meetings on the above days will be announced separately as soon as the full programme is definitely settled.

Regarding the Exhibition, the latest date for the reception of exhibits is Wednesday, June 23rd, 11 a.m.

The Organising Secretary is Mr. G. Montagu Harris, from whom all particulars can be obtained on application at Caxton House, Westminster, S.W.

Fires Due to Air Raids, etc.—As it is within the bounds of possibility that attempts may be made to start incendiary fires, the British Fire Prevention Committee make the following suggestions :—

(a) The various Fire Brigades cannot be expected successfully to deal with any large number of outbreaks, and their attention would in any case be primarily devoted to fires affecting Government offices, workshops engaged on Government work, military stores, docks and hospitals and the like, rather than to private property or semi-public property, however valuable. It therefore behoves all owners and occupiers of such premises to do their best to organise for self-help.

(b) Innumerable establishments such as factories, warehouses, stores, modern office blocks, flats, hotels, and large private houses, as also the semi-public establishments like schools, baths, theatres, music halls and assembly rooms are

equipped with internal hydrant services and first-aid fire appliances. These should be promptly examined and the staffs (both male and female) frequently practised in handling the gear. The smaller establishments, such as shops, offices and ordinary domestic buildings should have an ample supply of buckets (kept ready filled with water), and where funds permit, one or more small hand-pumps. Occupants should know how to use these appliances efficiently.

(c) Every effort should be made by the public to assist the Fire Brigades in their onerous duties, but they should not omit on any account immediately to notify any outbreak that comes to their notice by telephone or otherwise—the method of calling assistance to be posted up in a conspicuous position.

Concrete Buildings for Small Holdings.—Mr. T. P. Bonsor, F.R.I.B.A., who was awarded the prize by the Royal Agricultural Society of England for the design of house and buildings for a small holding, in a paper on the construction of such buildings says that, save from its æsthetic standpoint, concrete possesses even more advantages than brickwork, and, where gravel or other suitable aggregate is very cheaply procured, and the amount of work to be executed is sufficient to warrant the construction of the special framing required or the purchase of suitable appliances for making it in blocks, may be used with advantage.

Tests of Six Types of Reinforced Concrete Fence Posts.—Six types of reinforced concrete fence posts have been tested under the direction of the Committee on Signs, Fences and Crossings of the American Railway Engineering Association. All posts were made of 1 : 2 : 4 concrete, using broken stone or gravel aggregate not more than $\frac{1}{2}$ in. in size, and were about one year old when tested. Two round, one square, one half-round, one T-shaped, one triangular section were used, all 7 ft. long, all of tapering outline except the last. The reinforcement consisted of various sizes of steel wire or rods, and the lateral dimensions varied from 5 to $6\frac{1}{2}$ in. at the base to 3 or $4\frac{1}{2}$ in. at the top, reinforced with six No. 8 wires. The other smaller circular post reinforced with five wires also gave very good results. The tests indicated that this form of post, with several reinforcing wires, or the T-form with three $\frac{1}{4}$ -in. rods, was the strongest and most reliable.

The Effectual Sealing of Water Dams by Cementation.—In a paper recently read before a meeting of the National Association of Colliery Managers, at Leeds, Mr. Frank N. Waterhouse read an interesting paper on the above subject. The paper was reproduced in the *Iron and Coal Trades' Review*, from which journal we publish the following extract:—

In introducing his subject, Mr. Waterhouse said that at one of the collieries of which he had charge there were two seams being worked—the Old Hards or Parkgate, and the New Hards or Silkstone, the former lying at a depth of 116 yards from the surface, and the latter at 165 yards. On the north side of the Pit there was a 72-yard dip fault, which placed the Old Hards 22 yards below the New Hards. Another fault, which lay about 300 yards parallel to the first one, rose 23 yards. Two drifts were driven down the 72-yard fault to the Old Hards seam, the back drift at an inclination of 1 in 2, and the main drift at 1 in 6. A small area of coal was then worked out between the two faults, but as it proved very faulty it was decided to prove the rise faults. Accordingly a drift was driven at an inclination of 1 in 2, and then the main drift was driven in the rise at 1 in 6. The coal was opened out by headings, and water began to be very troublesome in roof, coal and floor. The seam was 20 in. in thickness, and had a fireclay seat.

A coal-cutting machine was installed and every effort made to get a good area of coal worked out. It was anticipated that the water would ultimately drain off. A pump capable of dealing with it was installed, but there was great difficulty in keeping up the roof in the face and the gates, and the quantity of water had steadily increased until it amounted to 60,000 gallons per day. It was therefore decided to close the district and dam back the water to avoid further pumping.

The first dam was built in the back dam drift, the site being prepared in the usual way by shearing the sides back and cutting in the floor and roof to secure solid ground. The main drift dam was then built; both were built with concrete and casing walls, the concrete being heavily reinforced with old wire ropes. When the valves were closed and the water allowed to fill up, the pressure gauge reading increased until it reached 50 lb. per sq. in. At this stage the dams held tight, but the

defective strata above referred to allowed the water to leak, the leakage being about one-half of the original feeder; quite two-thirds came from the back drift dam. All the time this leakage was allowed to continue the pressure of 50 lb. per sq. in. remained constant. It was then thought advisable to put another dam in the back drift. Pitchpine blocks, 5 ft. long by 10 in. square at the back, planed and tapered, were prepared on the surface and numbered. A level concrete bed was placed 9 ft. in front of the other dam, on which tarred flannel was laid to make the bottom joint in the ring, the joints between the blocks and roof and sides being made by ramming oakum with special tools. A 16-in. square cast-iron pipe was put in for a manhole and a 4-in. square pipe for the water to escape; these were made square so as to be able to fit the timber and make a tight joint.

An additional dam was built in the main drift 10 yards in front of the other, and it was hoped that the strata surrounding the new dams would prove water-tight. The water, however, dripped out in places two or three yards in front of the dam in the back drift, and the amount of leakage there was only slightly reduced. The dam in the main drift was good, except that at a short distance away there was a slight trickle of water in the floor. The advisability of adopting the cementation process of solidifying broken strata was then considered. A horizontal pump having a cylinder 10 in. diameter, 12-in. stroke, and a 2-in. ram, was used to inject the cement. Power to drive the pump was obtained by use of compressed air at a pressure of 75 lb. per sq. in. The suction pipe was connected to a tub in which the cement was mixed, the tub being fitted with a revolving arrangement which was worked by hand for stirring the water and cement.

Before pumping was commenced the 9 ft. of space between the two dams in the back drift was filled with broken bricks and stone. Three 2-in. pipes were laid through the manhole, rising one above the other. A thin iron plate was bolted on the manhole out-by end with the pipes projecting through the plate, the two upper pipes were plugged, and the bottom one connected to the delivery pipe of the pump. Cement and water were mixed in the tub until there was a mixture like thick mud. The pump was then started at 30 strokes per minute. After 40 hours' pumping the space was filled with cement, and the pressure gauge reading on the pump began to rise very quickly; the solution was then made thinner and pumping was continued until the pressure was 1,700 lb. per sq. in., when the pump stopped, not being able to force any more cement into the strata and the space between the dams. At the close of this work it was found that the whole of the operations had been successful, the water being completely stopped.

A wall was then built 18 in. thick in front of the dam, leaving a space of 9 ft., which was filled with broken stone, and cement was then injected in a similar manner as in the back drift.

During the progress of this operation the pressure produced by the injection of the cement affected the strata for some 6 yards on the out-by side of the dam, thus showing the fine fissure of the strata, and, on the other hand, the penetrating power of the fine liquid cement under heavy pressure.

At one period it was thought that it would be necessary to bore flank-holes in the strata, in order to give the cement direct access to the fine fissures. However, it was found in both cases that the injection behind the dams had proved successful.

In conclusion, Mr. Waterhouse suggested that, in the process of cementation, colliery managers had an additional ally with which to overcome difficulties of the nature he had described.

Nairobi Stock Exchange Buildings.—Mr. J. K. Watson, the general contractor, has completed the first portion of the Nairobi Stock Exchange, which occupies a position in the principal thoroughfare. The façades are faced with stone and backed with concrete, and all the interior walls are constructed of concrete. The Kahn reinforcement for the floors and flats, etc., was supplied by the Trussed Concrete Steel Co., Ltd., Westminster.

Expenditure on Public Works.—At the commencement of the war most of the local authorities of Great Britain looked round to see what works of public improvement might be taken in hand so as to cope with the unemployment that was feared, and it seemed as if many projects, in which concrete would form a large part, would be carried out. Scarcity of labour for the carrying out of Government work has caused

The BRITISH STEEL PILING CO

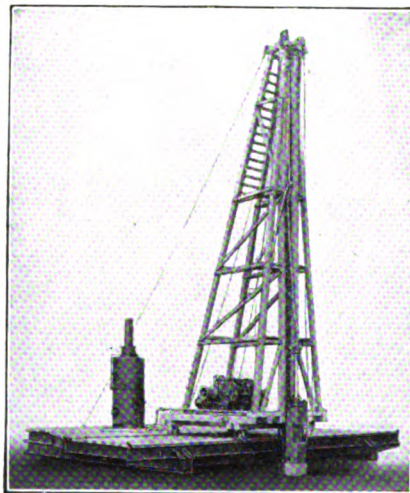


Illustration shows a Travelling and Traversing Steam Pile Driving Plant, built by us for the Crown Agents for the Colonies, for constructing a Reinforced Concrete Jetty at Lagos.

The Hammer is of our single-acting semi-automatic type, $2\frac{1}{2}$ tons in weight. The under-frame of the plant is of steel throughout, has an overhang of 12' 6", and travels on three sets of wheels. Telescopic leaders with ladder were supplied with the Pile Driving Frame.

This plant is of our standard type, so that the under carriage may be dispensed with and the plant used for ordinary pile driving work on completion of the Jetty at Lagos.

Weights of Piling in stock on sale :—

"SIMPLEX"	-	22 to 27 lbs. per sq. ft.
"UNIVERSAL"	-	from 43 lbs. "

DOCK HOUSE, Billiter Street, LONDON E.C.

the Treasury to restrict capital expenditure on these projects, with the result that many of them have been postponed.

LITERARY NOTE.

Messrs. Chapman and Hall have ready for publication a text-book on the "Strength of Materials," by Mr. Ewart S. Andrews, B.Sc., whose books upon the "Theory of Structures," published by the same firm, have had a wide circulation. The present book is based upon the same general lines, the two subjects having much in common, and comprises twenty chapters dealing with Stress, Strain and Elasticity; The Behaviour of Various Metals under Test; Repetition of Stresses, Working Stresses; Riveted Joints and their Pipes; Bending Moments and Shearing Forces on Beams; Geometrical Properties of Sections; Stresses in Beams; Deflections of Beams; Columns, Stanchions and Struts; Torsion and Twisting of Shafts; Springs; The Testing of Materials; Fixed and Continuous Beams; Distribution of Shearing Stresses in Beams; Flat Plates and Slabs; Thick Pipes; Curved Beams; Rotating Drums, Disks and Shafts.

PROPOSED NEW WORKS.

Patea (New Zealand).—In connection with the loan recently secured by the Patea Harbour Board, New Zealand, the Marine Department has approved plans for the improvement of the port, and tenders have been invited for the extension of the existing breakwater in concrete on the "pierre perdue" system.

West Hartlepool.—Messrs. W. Gray and Co., Ltd., contemplate the construction of two graving docks, a fitting-out basin, and a number of building berths on a new site acquired by them near Seaton Snook, at the mouth of the River Tees. Mr. I. C. Barling, of Middlesbrough, is the engineer.

Guernsey.—The St. Peter Port Harbour Committee, Guernsey, have invited tenders for the erection of a concrete tower upon the Roustel Rock, in the Little Russel Passage on the eastern side of the island, but no contract will be entered into until the registration of His Majesty's Order in Council authorising the execution of the work.

Alexandria.—The Egyptian Estimates for 1915 make provision for the reinforcement of the breakwater at Mole E and Quay R in Alexandria Harbour.

Clacton.—A Local Government Board enquiry has been held into the application of the Urban District Council for sanction to borrow £9,150 for works of sea defence. The proposal is to replace a further portion of the old timber wall with massed concrete. It will be 20 ft. in depth at the face and 21 ft. at the back; 9 ft. in width at the bottom and 3 ft. 6 in. at the top below the coping.

Port Talbot.—Considerable developments are anticipated at Port Talbot, and the Port Talbot Railway and Dock Co. are making preparations. Messrs. Baldwins, Ltd., have leased about 40 acres of land by the side of the Margam Dock for new works, and will construct a reinforced concrete wharf alongside the new works.

Caerphilly.—Tenders have been invited by the Urban District Council for the construction of a concrete retaining wall adjoining the Rhymney River at Pwllypart.

Reading.—The Town Council have instructed the Town Clerk to enquire of the Local Government Board if they will approve of the borrowing of the sum necessary for the construction of the De Bohun Road bridge in reinforced concrete.

East Africa Protectorate.—A Bill has been passed by the local Legislative Council providing for a loan of £1,868,000 from the Imperial Treasury to be expended on works in the Protectorate as follows:—Kilindini harbour works, £640,000; railway improvements, £957,000; roads and bridges, £100,000.

Brazil.—The city of Pernambuco, Brazil, has under construction an electric power house of reinforced concrete.

Withernsea.—During the month the Urban District Council have invited tenders for the erection of a reinforced concrete wall at the South Cliff of a length of about 450 yards.

Sunderland.—A House of Lords Committee has passed the preamble of the Bill of the Corporation for the reconstruction of Wearmouth Bridge at a cost of nearly £170,000.

Carlisle.—The Rural District Council have invited tenders for the construction in concrete of a water tank at Wetheral Shields.

Metropolitan Water Board.—The Board have decided to improve the coal storage bunkers at Hampton, and will carry the bunkers on a reinforced concrete pile foundation.

Eston.—The Tees Conservancy Commissioners have approved plans submitted by the North-Eastern Railway Co. for the construction of a line of deep-water quays about a mile in length at Eston. It is not expected, however, that the scheme will be proceeded with until the war is ended.

Concrete Houses at Linthwaite.—A report has been submitted to the Linthwaite Urban District Council by Mr. J. Ainley, the architect, on the scheme for the erection of concrete houses for the working classes. The scheme was for 89 houses at a cost of £26,000. Sixteen houses have been erected at a cost of £3,613, exclusive of land. Rents were yielding £196 per year; repayments of principal and interest amounted to £160, and the cost of repairs was £16.

CONTRACTS ACCEPTED.

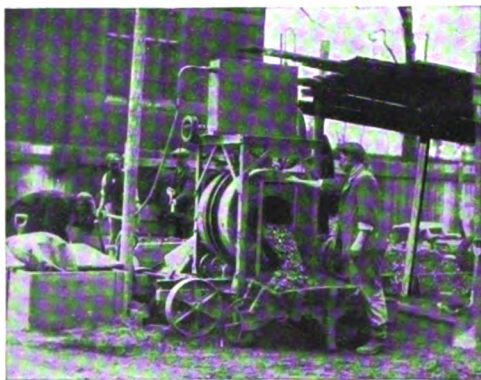
The Unit Reinforcement Construction Co., Ltd., have just received the following two contracts:—

950,000 gallons filtered water reservoirs, wash-out tanks, and lime pits at Blackmoorfoot, for the Huddersfield Corporation. J. W. Armitage, Esq., Water Works engineer.

Extensions to works for Messrs. Middlemosts Bros., Huddersfield. Messrs. Stocks and Sykes, architects.

CONTRACTS OPEN.

Minehead.—Tenders are invited by the Somerset County Council for the provision of a concrete apron of a length of about 750 ft.; the construction of a length of 120 ft. of new sea wall, and the construction of a reinforced concrete casing to Pill Bridge outfall. Specifications, etc., may be obtained of Mr. E. Stead, the county surveyor, Wells, Somerset, on a deposit of £3 3s. Tenders returnable on June 5th.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

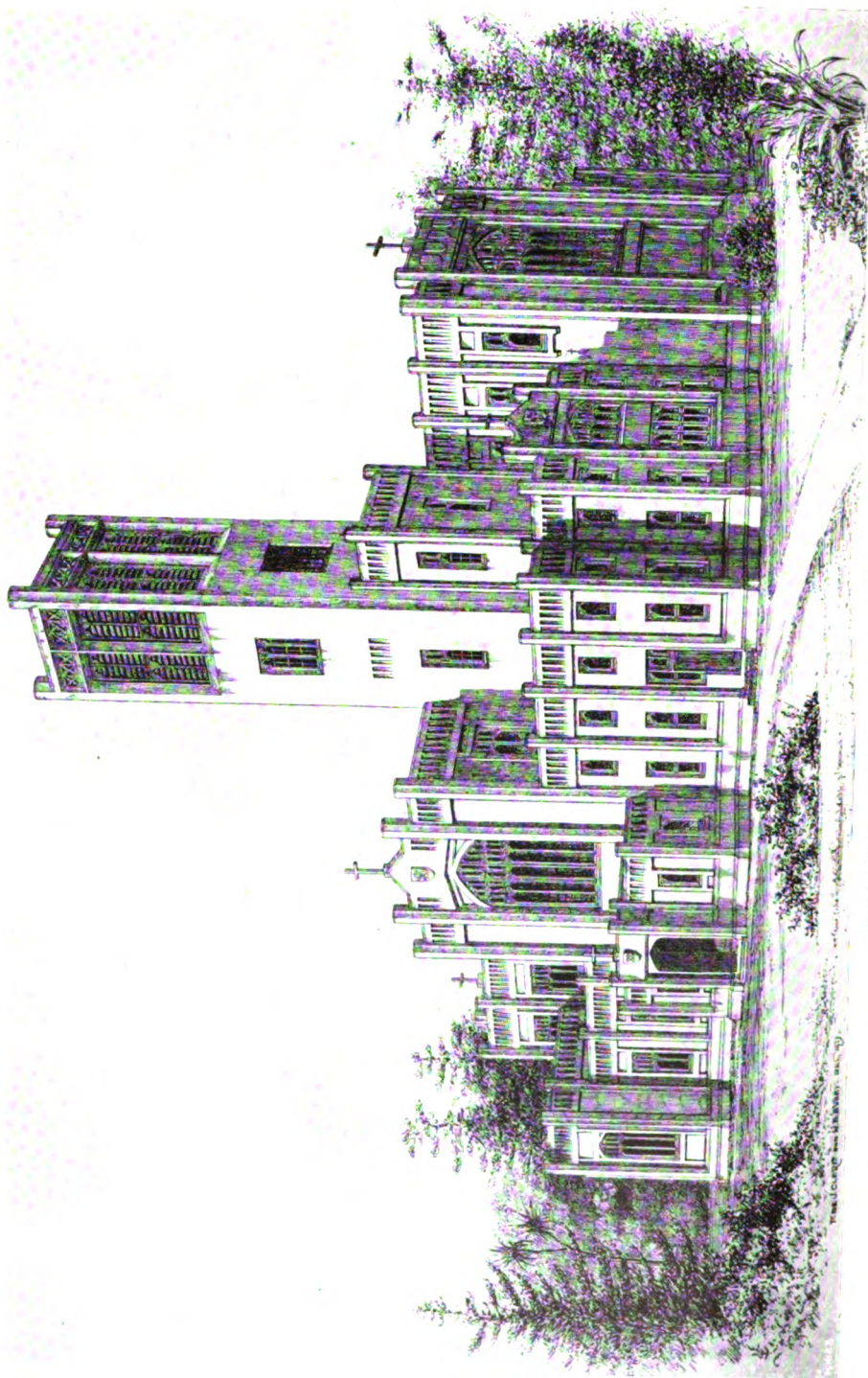
The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



REINFORCED CONCRETE CHURCH, HASTINGS, NEW ZEALAND.
See page 372.

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X., No. 7.

LONDON, JULY, 1915.

EDITORIAL NOTES.

CONTRACTORS AND REINFORCED CONCRETE.

THE existing state of affairs as relating to the general building contractor and the tendering and execution of reinforced concrete leaves much to be desired, from the contractor's and the designer's point of view, and it is in the hope of arousing some interest in the conditions that we put forward our remarks.

The consulting engineer and the specialist are in the hands of the contractor when the estimating and execution come to be considered, and it therefore follows that with the most careful calculations and quantities a great variation of price will result with different contractors, and even with the most careful supervision errors are liable to occur if the builder is not well versed in the nature and properties of the material which he is using.

It is true that a limited number of contractors have priced and executed reinforced concrete to a large extent, and these are able to produce satisfactory results; but the majority of builders have had little or no experience, and are really not competent to enter the market as competitors under the present conditions. Apart from special licensed contractors, and specialists who execute the work themselves, the general method now adopted when reinforced concrete forms the whole or part of the constructional material in a building is for the engineer, whether specialist or otherwise, to prepare a scheme with the necessary details from which the quantities are taken out, and the bills are then sent out to several builders for quotations. If the bills form part of the quantities for the whole building, then the tenders obtained are for the whole of the structure and finishings, and when the reinforced concrete is dealt with by a specialist firm apart from the building generally the lowest estimate is introduced as a provisional amount in the main contract. In the latter case it follows that, if the lowest estimate for the reinforced concrete is not given by the contractor who is also lowest in the building tenders, two separate contractors will be employed, and this is not conducive to economy or speedy erection, and it is preferable to adopt the former method in which the reinforced concrete quantities form part of the bills for the whole building. In order to do this satisfactorily, however, it is necessary either to invite tenders for the whole building from contractors who have had special experience in reinforced concrete, or ask for prices from various firms, regardless of previous experience, and thus run the risk of an inexperienced firm being the lowest.

In a recent case, when the bills for the reinforced concrete work, which formed about one-third of the whole work in the building, were included in the general bills, some very good tendering was obtained, but it was found that the total price of the reinforced concrete work varied more than 40 per cent.

in the highest and lowest tender, and this in spite of the fact that the quantities were most carefully prepared, and the contractor had no speculative items to deal with. The architect wished to invite some of the builders who had done building work generally for him for many years, and the list included some who had little or no previous experience in reinforced concrete work. Some of the contractors actually expressed their doubts and uncertainty in preparing their prices even with such complete quantities, and this feeling was caused simply because they were dealing with a material which they did not understand. The remedy is really in the hands of the contractors themselves, as it behoves them to study the material and its properties, and thus they could deal with it in an intelligent manner. We do not mean that every contractor should devote years of study to the theoretical design of the material, but he should study it sufficiently to understand those principles which govern the design and disposition of the component parts, he should be able to appreciate the relative values of the steel, concrete and centering. How is it possible for a builder to carry out successfully any scheme when he is working in a material with which he is not in sympathy? It will be found that it is only those who do not understand the material that have so much to say against it, and it is a positive fact, proved by long experience, that when the subject is studied, and the theoretical and practical principles are thoroughly understood, then and then only do designers and builders become enthusiastic about the use of the material.

If several contractors were called upon to price an ordinary bill of quantities of constructional steelwork, would there be a difference of 40 per cent. in the tenders? It is very unlikely, and it would also be incorrect to say that any one of them would be dubious about executing the work; but the same remarks do not apply to reinforced concrete. The ordinary contractor would probably not hesitate to calculate a simple beam or column if he were asked to give a suggestion in a steel building, but he would be quite incapable of doing the same thing in the case of reinforced concrete, and that is where the fault lies. It is not that one wishes the builder to design or give suggestions, but is only an illustration of his knowledge of one form of construction and his lack of knowledge in the case of the other.

This is the age of science, and reinforced concrete being essentially a scientific material is bound to become more universal every day, and unless those responsible for the execution and pricing keep abreast of the times, and study the scientific application of a scientific material, they will be left behind and their interests will suffer.

The builders of the next generation will look upon reinforced concrete in a very different manner, simply because the subject will be included in their training; but there is no reason why the present-day contractor should not make himself proficient, and the architect and specialist would then feel they were obtaining tenders from one who was conversant with the material.

The application of reinforced concrete has been greatly hampered in the past by the attitude of architects and contractors who fail to appreciate the advantages of the material, but whereas the former can generally make a choice in the matter, the latter have none, and they must either price and execute in a satisfactory manner or lose a large portion of their work in the future.



View of Back Wall of Bridge.

REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

By CHAS. E. HOLLOWAY, B.Sc.

This bridge is entirely of Reinforced Concrete, and affords an interesting example of the application of this material to braced girders of considerable span and abutment walls of large proportions.—ED.

Introduction.—The bridge described in this article was built for Messrs. Nixon's Navigation Co., Ltd., of Merthyr Vale, under the direction of Mr. W. T. G. Marsh, their engineer, to convey the refuse colliery shale from their Merthyr Vale pit to the new site which they have acquired for their spoil tip.

As seen from *Figs. 1 and 2*, the Rhymney railway line and the Glamorganshire Canal run side by side between the pit workings and the new tipping site on the mountain, the new bridge carrying the track for the spoil trams across these obstacles. At this point the railway and canal are about 80 ft. apart and

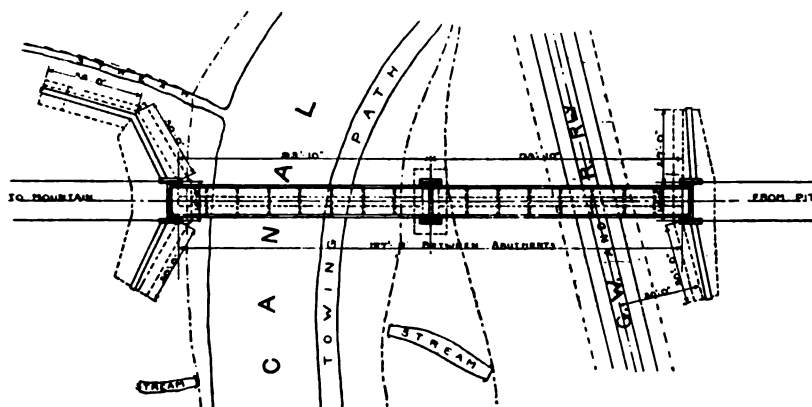


Fig. 1. General Plan.

REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

run on embankments of considerable height, and the general outline of the bridge is governed by the following requirements:—

- (a) Height from rail level of Rhymney railway to under-side of girders at lowest point is 20 ft.

- (b) Gradient of bridge deck 1 in 18 towards mountain.
- (c) Distance from centre of railway to face of railway abutment is 30 ft.
- (d) Face of canal abutment is at canal fence.
- (e) The abutment walls retain the embankments of colliery spoil, which form the approaches to the bridge.

The distance between the abutments in the clear is 187 ft. 8 in., and the

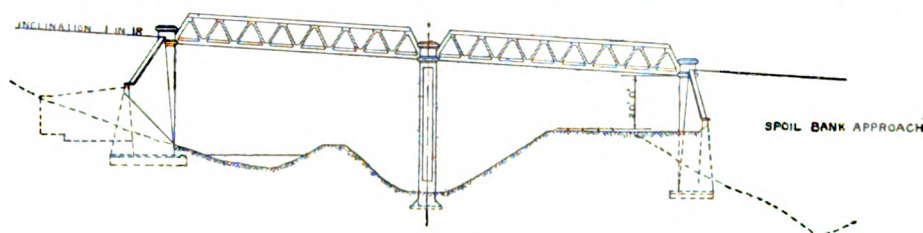


Fig. 2. General Elevation.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.



Fig. 3. View of Completed Bridge.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

heights of the abutments from foundation to formation level are 44 ft. and 46 ft. respectively for the canal and railway sides.

A scheme was originally prepared for a bridge with masonry abutments similar in outline to *Fig. 4*, with three steel plate girder spans supported by two masonry piers. The amount of masonry involved was very great, as the abutment walls were to have been 10 ft. thick. This was undesirable, firstly because it would have been very costly to get so much material to a site so

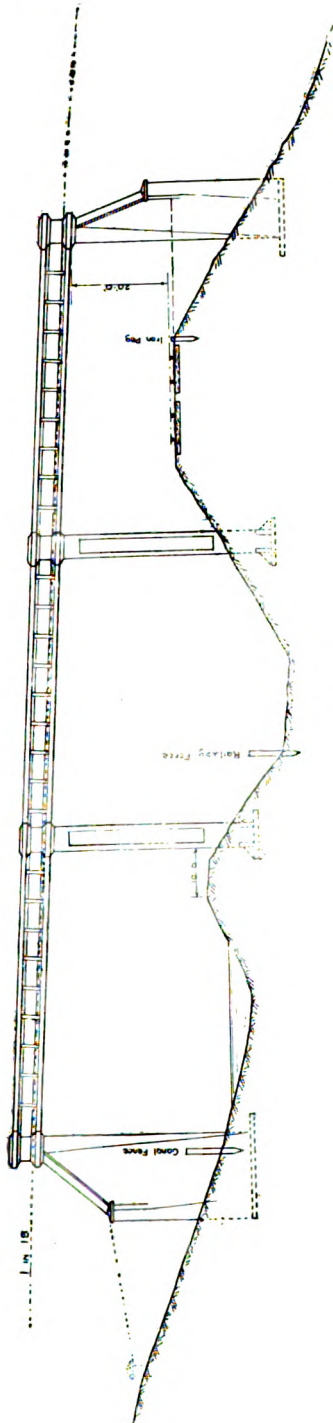


Fig. 4. General Elevation of Bridge as Originally Intended.

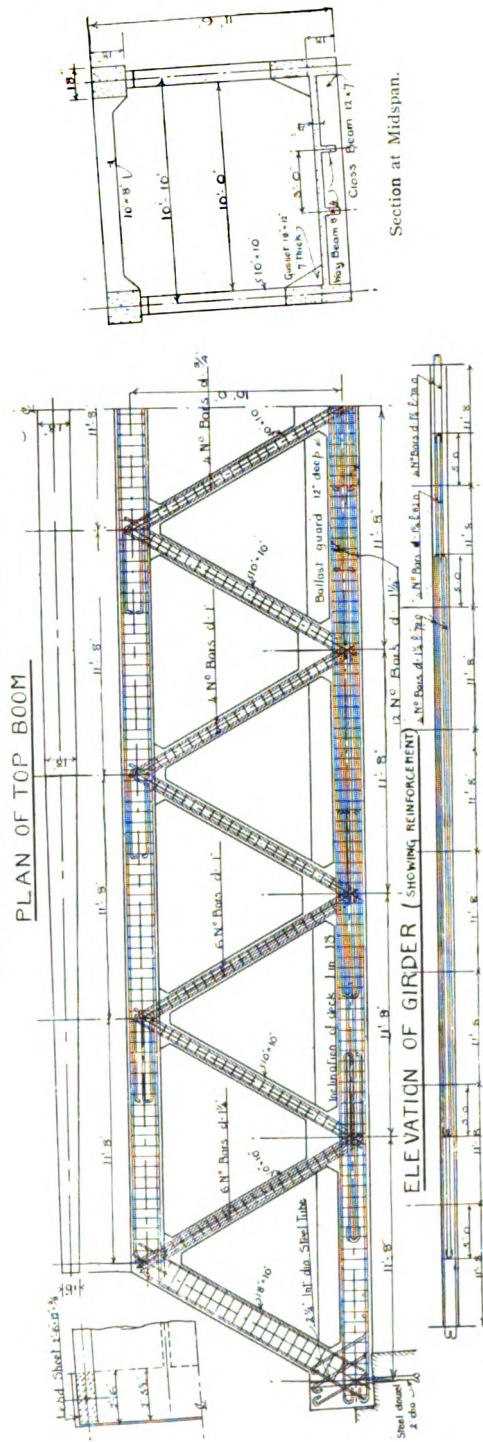


Fig. 5. Details of Girders.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

difficult of access, and secondly because the ground in this locality is very unstable owing to the workings underground.

The attention of the owners was eventually directed to reinforced concrete as the suitable method of construction owing to its lightness and elasticity, and the scheme illustrated in *Fig. 4* was selected, being the tender of Messrs. S. Robertson, Ltd., of Bristol, on designs and specifications prepared by the author.

This scheme, which embodied three solid-webbed reinforced concrete girder

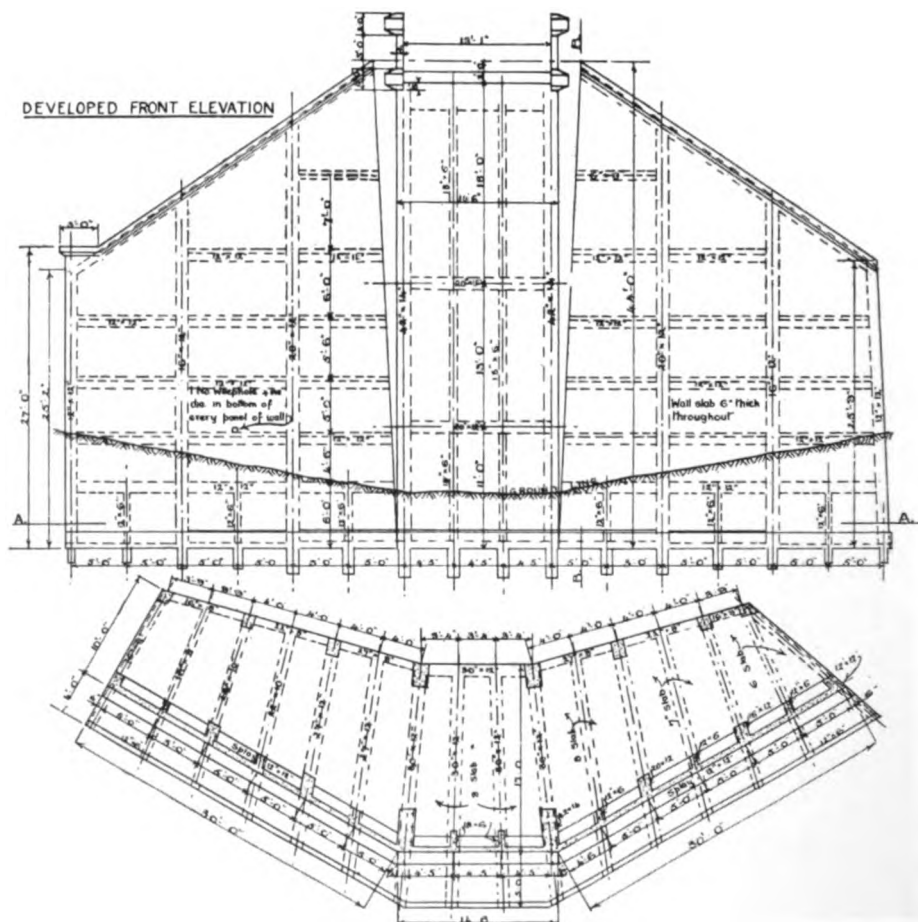


Fig. 6. Elevation and Plan of Canal Abutments.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

spans, was finally altered to the two-span braced Warren girder bridge shown in *Fig. 2*, owing to a dispute about the pier resting on the canal bank. *Fig. 3* gives a general view of the finished structure.

Abutment Walls.—The abutment walls are of very light design and show a large saving over other methods of construction. The erection is somewhat more complex, but presents no difficulty if properly organised and carried out by men experienced in this class of work.

Fig. 6 gives an elevation and plan of the canal abutment walls, and *Fig. 7* shows a section near the centre line of the bridge. A view of the back of this wall is also given in the title illustration. Under the girders the wall is 44 ft. high and 14 ft. 6 in. wide, and is vertical. The wings extend for 30 ft. on each side, and are 27 ft. high at their extremities. They are battered back at 1 in 12, and slope back from the vertical face at an angle of 30° in plan.

The walls are finished *Figs. 2 and 7*, the general effect being very satisfactory. The wall slab is 6 in. thick throughout, supported by horizontal beams stretching between the counterforts, spaced according to the calculated earth pressure. The slab is reinforced horizontally and vertically on both faces with $\frac{3}{8}$ -in. and $\frac{1}{4}$ -in. bars from $4\frac{1}{2}$ in. to 12 in. apart, and is well provided with weep-holes. The horizontal beams are specially designed to resist the cantilever action produced by the downward pressure of the filling, and account of this action had also to be taken in designing the slab.

The counterforts, as illustrated in Fig. 7, are of the open braced type designed as framed cantilevers about the base slab and supporting the wall slab, which resists the lateral earth pressure. This type of counterfort represents a considerable saving over the solid type. The bracings for these counterforts are 14 in. by 14 in. under the girders and 12 in. by 12 in. elsewhere, substantial gussets reinforced with $\frac{1}{2}$ -in. bars being formed at all angles. As seen from Fig. 7, the main tensile reinforcements in these bracings all spring from anchor blocks in the foundation slabs, where they are hooked around special bars placed under the foundation beams. These main bars branch off into the various bracings according to the forces which have to be resisted, and terminate near the face of the wall with ample return hooks. Many of

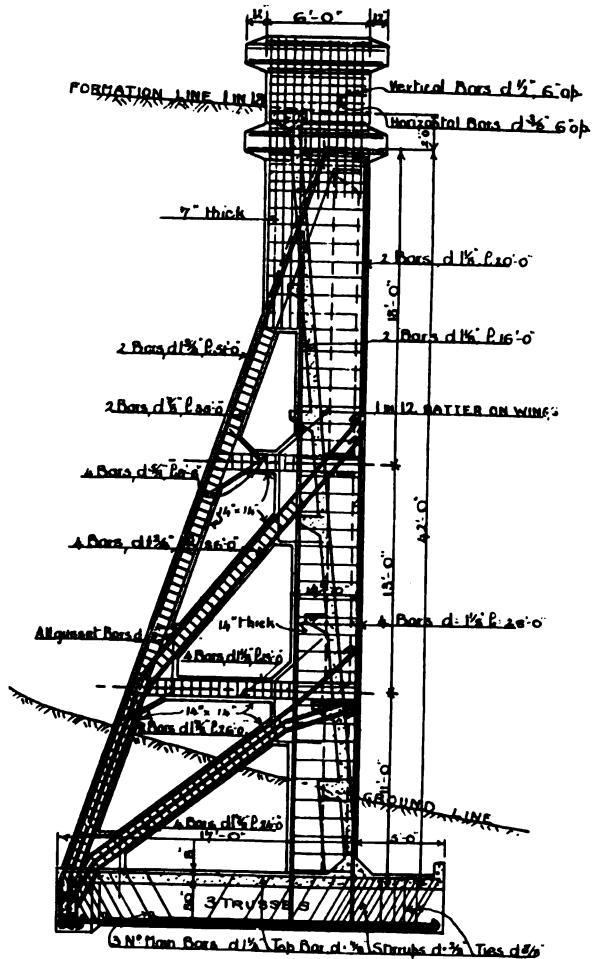


Fig. 7. Section Showing Reinforcement.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

these bars are of large diameter and length, and, after being bent to shape, were erected and secured in place by the scaffolding, which was erected

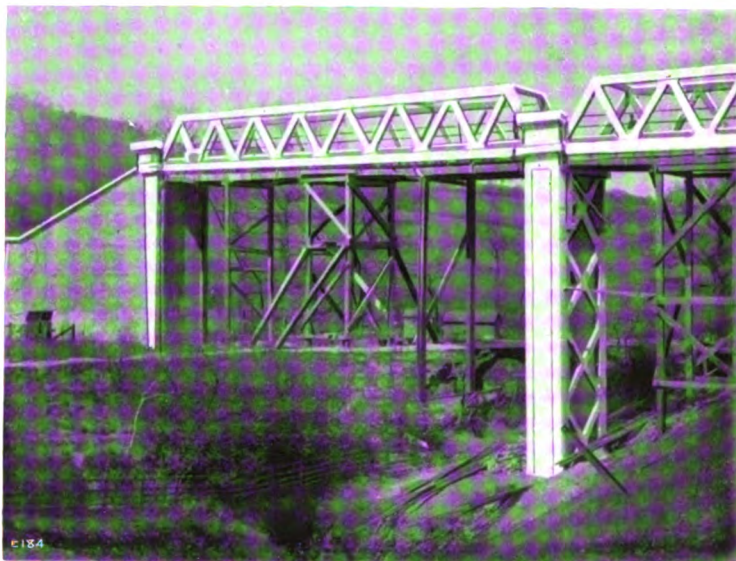


Fig. 8. View Showing Finished Walls.



Fig. 9. View Showing Girder Spans.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

beforehand for this purpose, and was afterwards also utilised for concreting and shuttering operations.

The foundation slab as illustrated in *Fig. 6* is 22 ft. wide under the girders, tapering to 14 ft. wide at the extremities. The toe projects 5 ft. beyond the face of the wall at the centre and 4 ft. at the wings. The slab varies from 8 in. to 6 in. thick, as shown, and is reinforced on both faces in both directions with $\frac{1}{2}$ -in. and $\frac{3}{8}$ -in. bars. It is supported at short intervals by beams running from toe to heel, which vary from 30 in. by 12 in. to 16 in. by 8 in. The reinforcement

for these beams at the centre is shown in *Fig. 7*, and consists of 3 unit trusses placed side by side with three No. $1\frac{1}{4}$ -in. main bars in each truss. At the heel these transverse beams are supported by a main beam, which runs into the anchor blocks, and at the toe they support a secondary beam, which carries the outer edge of the slab. The upward earth pressure at the toe tends to lift this beam and the slab off the transverse beams, and special reinforcement is provided to resist this action.

The abutment walls were designed to resist earth pressures in accordance with Rankine's theory, taking the weight of filling as 100 lb. per cu. ft. with an angle of repose of 36° . The resultant force on the foundations was made to fall within the middle third, and the factor of safety against overturning was calculated to be not less than 2. The girders were not assumed to assist in resisting the overturning forces, although their dead weight was taken into account when finding the resultant force on the base.

A stream flows into the canal near one side of the

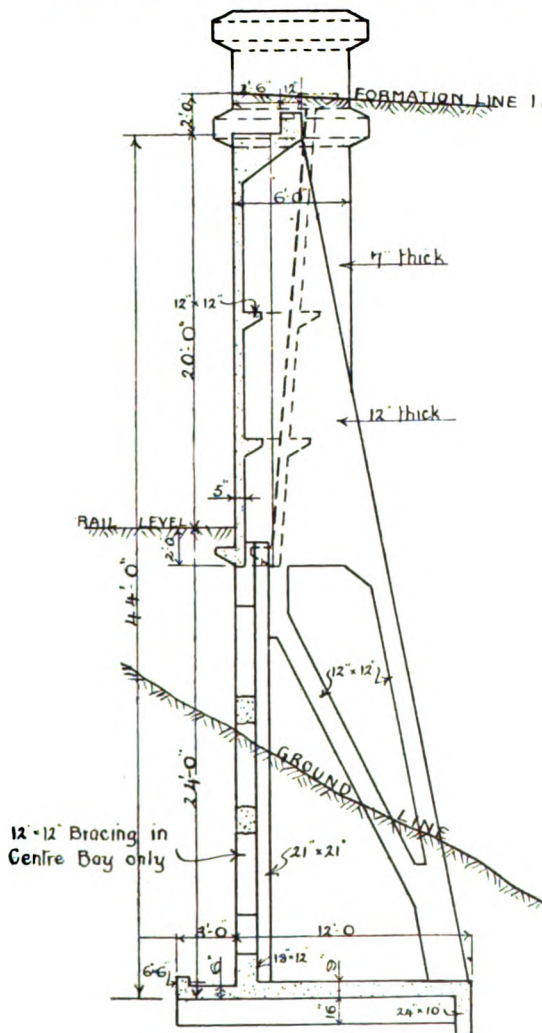


Fig. 10. Section of Railway Abutment.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

canal abutment, and a wing wall, 36 ft. long, is provided to retain the embankment at this stream, as shown in *Fig. 1*. This wall is 19 ft. 9 in. high where it is bonded into the canal abutment, reducing to 14 ft. at the other extremity. It is very similar in design to the other walls, but the counterforts, which are 9 ft. apart, are solid, 8 in. thick, as it was not found economical to construct counterforts of the braced type for this height of wall.

Railway Abutment.—The railway abutment embodies some interesting features owing to the fact that it will be buried up to the Rhymney railway formation level when the approach embankment to the bridge is made. From Fig. 10 it is seen that the vertical slab is omitted for the portion of the wall to be buried, thus saving a large amount of material and greatly reducing the proportions of the abutment owing to the resulting reduction in the lateral pressure on the vertical slab. The details of this abutment are very similar to those of the canal abutment. The foundation slab is 15 ft. wide at the centre and 9 ft. wide at the extremities, and varies from 9 in. to 6 in. thick, supported by beams as illustrated in the figure. Cross-bracing is introduced in the centre bay under the vertical slab to resist lateral forces such as wind-pressure, as this abutment is entirely supported on posts for the lower 24 ft. of its height, these posts to be eventually buried up as described above.

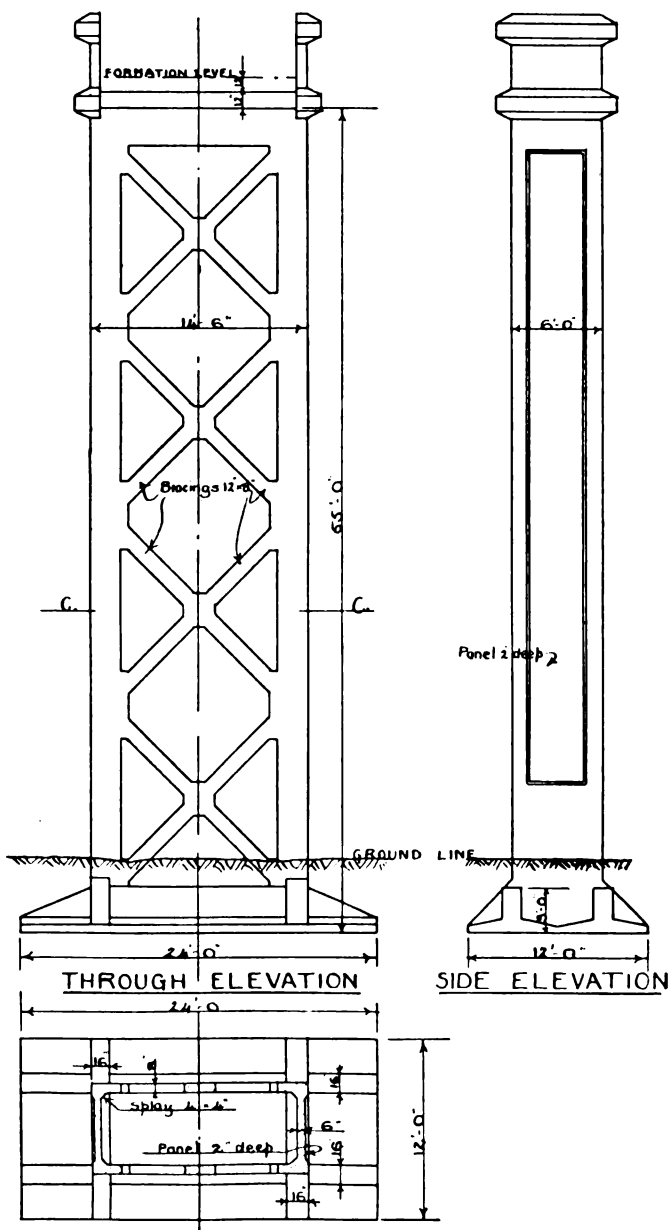


FIG. 11. Plan on Line C.C.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

The Pier.—The pier, as is the case with the other parts of the structure,

is of very light and economical design. It is well illustrated in *Figs. 11 and 8*. The foundation slab measures 24 ft. by 12 ft. and is 9 in. average thickness, reinforced with $\frac{3}{8}$ -in. and $\frac{1}{2}$ -in. bars. It is stiffened with beams 36 in. by 16 in. in both directions. The pillar measures 14 ft. 6 in. by 6 ft. It is braced diagonally with 12-in. by 8-in. bracings on the wider faces, and is solid on the other faces, being everywhere 8 in. thick except where it is reduced to 6 in. thick by the bevelled panels, 4 ft. wide by 2 in. deep, on the solid faces, as shown in the section, *Fig. 11*.

At the top of the pier under the girders are two beams 30 in. by 16 in., connected by a heavy slab to form a bearing.

The Girders.—The girder spans are of the Warren type, 93 ft. 4 in. centre to centre of bearings, and are illustrated in detail in *Figs. 5 and 9*. The girders are 11 ft. 6 in. deep over all and 10 ft. apart in the clear. The tension booms

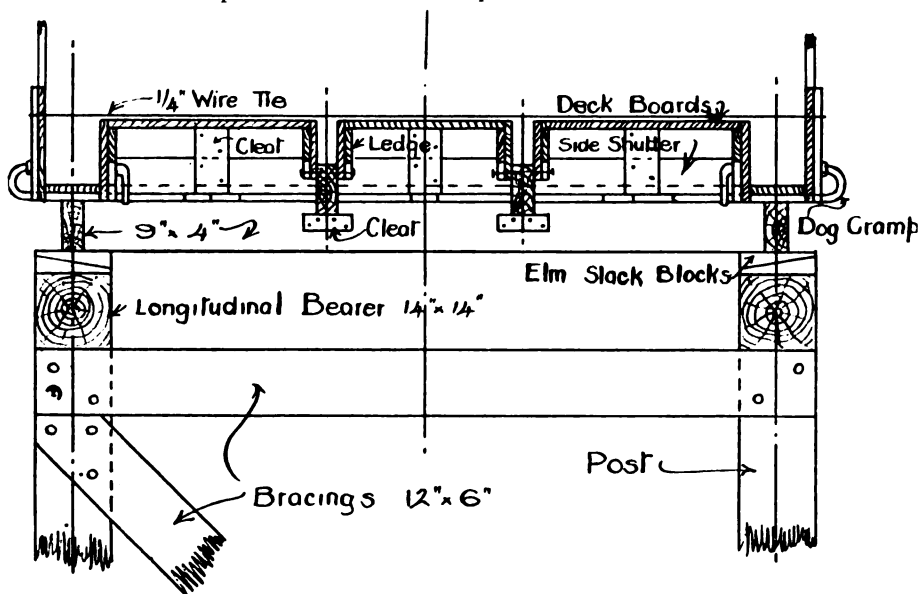


FIG. 12. Cross Section of Shuttering.
REINFORCED CONCRETE BRIDGE NEAR MERTHYR.

are 18 in. by 10 in., and the compression boom 18 in. by 18 in. at the centre tapering to 18 in. by 10 in. on the two end bays each side. The bracings are very light, the end posts being 18 in. by 10 in. and the remainder 10 in. by 10 in.

The tension booms are reinforced with 12 No. $1\frac{1}{4}$ -in. bars arranged in two sets of six, as shown in *Fig. 5*, the bars overlapping in such a way as to fit the bending moment diagram. In each case they extend 5 ft. beyond the last panel, which they are intended to reinforce and terminate in a return hook of 6 in. internal diameter. Each set of six bars is bound together laterally with $\frac{1}{2}$ -in. ties 6 in. apart except at the hooks where they are increased to $\frac{3}{4}$ in. The method of fixing this reinforcement in place is worthy of note. One set of six bars was arranged on wooden blocks laid on the deck shuttering. These bars were fixed at the correct distances apart by wedging wooden spacers between them. The ties were then placed on the bars and wired to them at

alternate intersections with 16 gauge annealed wire. A frame of reinforcement was thus made of sufficient rigidity to be bodily lifted and laid in the moulds.

The reinforcements for the tension diagonals were dealt with in a similar manner. As seen from *Fig. 5*, these bars are carried 4 ft. horizontally into the booms and the ends hooked at right angles, to secure the necessary bond. At the end joint in the top boom this bond cannot be secured, and the bars are anchored by hooking over the top reinforcement of the other members and returning the hooks a sufficient distance. Special attention had to be given to the joint at the bearing for a similar reason. The bending of these bars was done cold on a bench consisting of one timber 20 ft. long and 12 in. square on which were mounted the necessary studs. Heavy weldless tubes to slide over the bars were used as levers. The reinforcements for the compression members were wired into units on templates.

In each span there are 4 No. transverse wind braces 10 in. by 8 in. reinforced with 4 No. $\frac{3}{8}$ bars and $\frac{1}{2}$ -in. ties 6 in. apart. The bridge is also stiffened against lateral forces by 18 in. by 12 in. gussets connecting the diagonal bracings with the crossbeams.

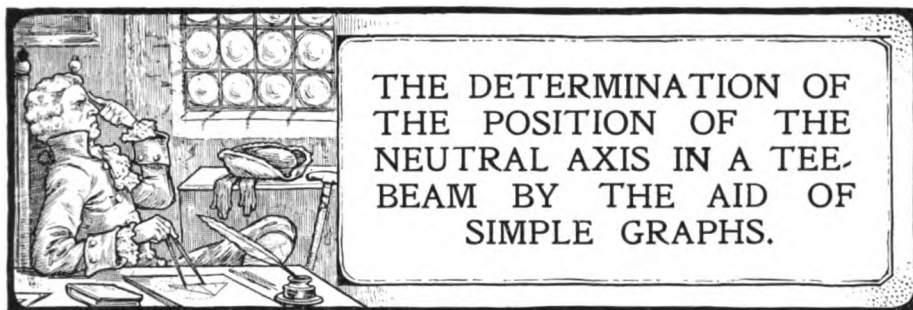
Owing to the possibility of settlement in the foundations the girders had to be constructed quite separate from the abutments and pier in such a way that they could be raised upon packing if necessary. As shown in *Fig. 5*, the main girders rest on lead sheets 30 in. by 12 in., $\frac{3}{8}$ in. thick. A steel dowel 2 in. in diameter projects up from the bearing into a 2 $\frac{1}{4}$ -in. internal diameter tube bedded in the girder, thus forming a bearing free to move vertically.

The shuttering for the girders was supported by strong timber trestling shown in *Figs. 1* and *8*, the verticals consisting of full timbers and the diagonals of half timbers. The trestling was bolted together with 1 $\frac{1}{2}$ -in. bolts provided with plate washers, and was erected with the help of a derrick pole and hand winch with blocks and tackle. The largest timbers used were 40 ft. long and 14 in. square. This trestling supported longitudinal timbers running through the bridge directly under the main girders, on which the shuttering was carried. *Fig. 12* shows a cross section of the shuttering adopted for the girders.

Generally.—The bridge was designed for a superload of 170 lb. per sq. ft. live and 50 lb. per sq. ft. for ballast.

The bridge spans have not yet been tested, as the approach embankment is not yet formed. The measured deflection of the span over the railway on taking its own weight when striking the slack blocks was $\frac{1}{8}$ in., which appears to show that the girders are very stiff.

The concrete for the abutments and pier was mixed in the proportions of 1 bag cement (11 bags per ton), 4 $\frac{1}{2}$ cu. ft. sand, 9 cu. ft. chippings. For the girders a richer concrete was used—namely, 1 bag cement (11 bags per ton), 3 $\frac{1}{2}$ cu. ft. sand, 7 cu. ft. chippings. Crushed pennant of very close texture was used for the chippings and crusher dust from the same stone was used for sand. A very satisfactory concrete was obtained, which also gave an excellent face to the work.



By H. E. LANCE MARTIN, B.Sc.Eng. (Lond.), of the University of Liverpool.

The following article will no doubt be of interest to all engaged in the designing of reinforced concrete.

FOR the proper design of a tee beam in reinforced concrete it is essential that the position of the neutral axis should be found. Unfortunately the recognised rules as issued in the second report of the R.I.B.A. and the Regulations of the London C.C. are somewhat complicated, and to anyone not skilled in using such formulæ these present some difficulties.

The experience of the writer as a lecturer on this subject more than bears out this statement, and he feels that a method which simplifies such is more than justified.

The formula as given in the report is $n_r = (s_r^2 + 2mr) / (2s_r + 2mr)$ and should be used only when the neutral axis comes below the slab.

n_r = The ratio of the depth of the N.A. n below the floor level to the effective depth d of the tee beam.

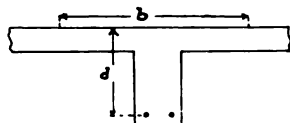


FIG. 1.

s_r = The ratio of the total depth d_s of the floor slab to the effective depth d of the tee beam.

m = The ratio of the moduli of steel and concrete = 15.

r = The ratio of the steel area A_s to the area of the concrete $b \times d$ in the tee beam.

Plotting this equation for various values of r , the relation between s_r and n_r is obtained (Fig. 2), in which six curves are indicated. Now it is at once seen that any one of the particular values of r chosen for this purpose will probably not be the value employed in any given design. Thus for a given case s_r and r are known, but n_r is unknown. Now the relation between r and n_r can easily be found from Fig. 2 by keeping the value of s_r constant—that is to say, by drawing a horizontal line parallel to the value of n_r , through the known value s_r , and reading off the values of n_r where this horizontal line cuts the curves. These plotted against the various values of r give the relation between all values of n and r , and a series of these is indicated in Fig. 3. These latter are more direct in use than those shown in Fig. 2, but care must be exercised in not confusing one curve for another in the places where they cross each other. They have been drawn with differently dotted lines in order to avoid this small difficulty as much as possible.

An example will make the methods of using these curves clear. Say

$d_s = 7"$, $b = 105"$, $d = 44"$, and $r = 0.00273$, all these would be known from previous calculations. Then $s_r = 0.159$. Now, using the curves of Fig. 2, follow horizontally the value of s_r along until the curve for $r = 0.002$ is reached. The

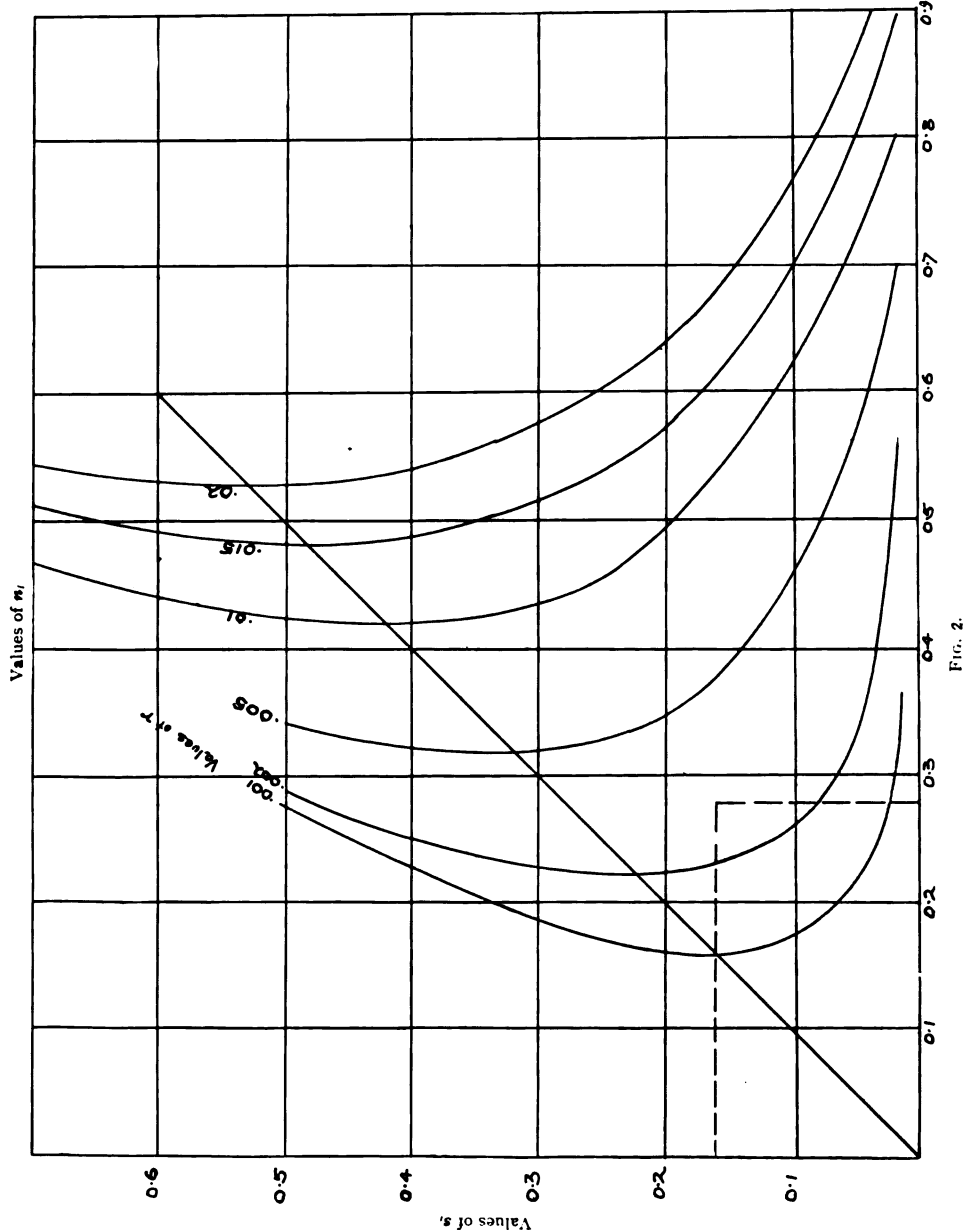


FIG. 2.

required point lies on this horizontal line between the curve for $r = 0.005$ and the above curve. Interpolate and read off on the n_r scale by following the vertical line down through the estimated point. Doing this we find that $n_r = 0.28$. The process is indicated by arrows in the figure.

Now, using Fig. 3, follow up vertically the given value of r until it reaches the curve for $s_1 = 0.2$; the required curve lies between this and that for $s_1 = 0.1$. Interpolate and read off the required value of n_1 by tracing the horizontal line drawn through the intersecting point. In this case $n_1 = 0.27$. By actually calculating from the formula $n_1 = 0.26$, which shows that the above values are quite near enough for practical purposes. If, however, this latter result is

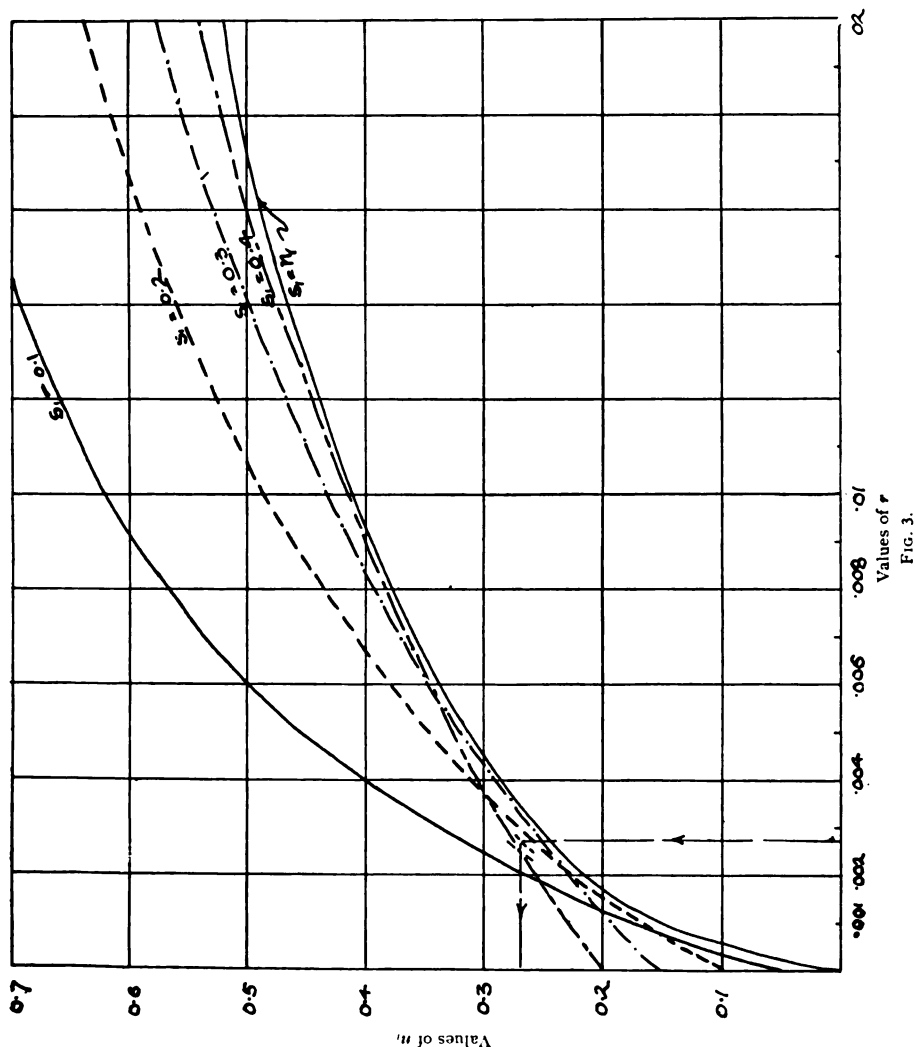


FIG. 3.

required from the curves it can be obtained by drawing the horizontal line in Fig. 2 through the given value of s_1 , so that it cuts all the curves and noting the values of n_1 corresponding to the intersection points; these values plotted against the values of r will give the exact curve which in the method above was only estimated. Now from the given value of r —viz., 0.0273—the corresponding value of n_1 can be read off directly on this new curve—viz., 0.268—the same

as given by calculation. Another very useful property of *Fig. 2* is that by joining all the minimum values of n_i together a straight line is the result, which passes through the origin. Now the minimum value of n_i is obviously when the neutral axis coincides with the underside of the floor slab, in other words when $n_i = s_i$. Looking at *Fig. 2*, this is at once seen to be the case; hence by drawing this line it can be determined whether the neutral axis is below the floor slab or not by noticing whether the estimated point lies with respect to this line.

This can be shown also mathematically by differentiating the equation $n_i = \frac{s_i^2 + 2mr}{2s_i + 2mr}$ and determining the minimum value n_i thus,

$$\frac{dn_i}{ds_i} = \frac{2s_i(2s_i + 2mr) - 2(s_i^2 + 2mr)}{(2s_i + 2mr)^2}$$

putting the numerator = 0 we have that $s_i = \sqrt{m^2r^2 + 2mr} - mr$, which if substituted in the original equation will give the required minimum value of n_i . This latter step, however, need not be done, as it will be seen that the value of s_i found above is exactly the same expression as that given in the report for n_i when the neutral axis falls within the slab, hence $s_i = \sqrt{m^2r^2 + 2mr} - mr = n_i$, the same result as obtained by considering the graphs.

Finally by plotting the various values of r in *Fig. 2* against the values of n_i obtained by noting the points of intersection of the curves and the slanting line, the relation between these is found when $s_i = n_i$ or, in other words, the position of the neutral axis is known with regard to r when this falls inside the floor slab.

Example 1.—Say $d_s = 5\frac{1}{2}"$, $b = 64"$, $d = 15\frac{1}{2}"$, and $r = 0.00675$. Then $s_i = 0.355$ and from the curves $n_i = 0.36$, $\therefore n = 0.36 \times 15.5 = 5.6$, showing that the N.A. in this case is just on the inside of the slab, which can be seen from *Fig. 2*, the intersection point being nearly on the limiting line.

Example 2.—Say $d_s = 5\frac{1}{2}"$, $b = 81"$, $d = 22"$, and $r = 0.011$. Then $s_i = 0.25$ and from the curves $n_i = 0.47$. The intersection point in this case is well below the limiting line, therefore the N.A. should be below the underside of the slab, its actual position being $n = 0.47 \times 22 = 10.3"$.

Example 3.—Say $d_s = 6"$, $b = 60"$, $d = 21"$, and $r = 0.0025$. Then $s_i = 0.285$ and from the curves $n_i = 0.24$. The intersection point is, however, above the limiting line, therefore the curve given in *Fig. 3* must be used. The position of the N.A., however, using $n_i = 0.24$, works out to be $0.24 \times 21 = 0.285$, which is inside the floor slab. Now, using the proper curve $n_i = 0.235$ therefore $n = 4.9$, a result very near to that given above. It is to be noticed that for the purpose of the equation $n_i = (s_i^2 + 2mr)/(2s_i + 2mr)$ when $s_i = n_i = \frac{n}{d}$ that the above value of s_i —viz., 0.285 —is incorrect, the proper value being $\sqrt{0.235}$, which is the same value as found above. In other words, when the N.A. falls within the slab the ratio of n to d must be used (see *Fig. 4*), instead of d_s to d .

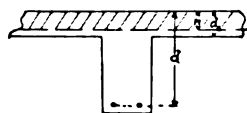
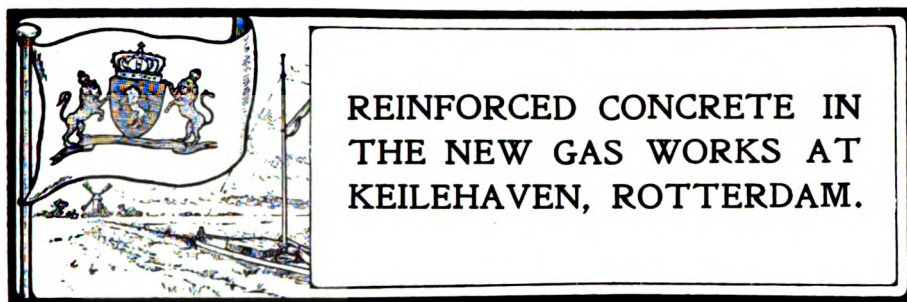


FIG. 4.



We are indebted for the following particulars and photographs to Mr. H. C. Stissingh, the Director and General Manager of the Rotterdam Gas Works.—ED.

REINFORCED concrete has entered very largely into the construction of the buildings for the new gas works at Keilehaven, Rotterdam. These works were started in 1913, and are so arranged as to allow for future extensions.

In view of the unstable soil in Rotterdam it was necessary to place all the buildings on pile foundations, and in consequence of the surface water level the tops of the piles are brought up to a point 3·25 m. below the floor. This has led to the provision of watertight cellars to the greater part of the buildings, in order that they could be used, in connection with the installations, for gas mains, conduits, or other apparatus. These cellars are also constructed of reinforced concrete.

Where there was no necessity for underground accommodation the foundations are built solid with reinforced concrete walls or columns and ties. This is shown in *Fig. 1*, indicating the foundation of the retort house and the coal bunker house annexe.

Reinforced concrete was also used for the underground tar and ammonia-water tanks, and for various floors, thus forming the main constructional material for a portion of the buildings of this new water-gas plant.

The accommodation of this portion includes a cellar, through which the main conduits run, an exhauster house, a blower-room, and a flat rectangular water tank for cooling off the wash-water. This building can be seen in the centre of the background in *Fig. 4*, and it is also just indicated in the left-hand corner of *Fig. 5*. The panels between the external reinforced concrete framing where not utilised for windows are filled in with brickwork.

Special mention should be made of the foundation of a gasholder (of 75,000 cu. m. capacity), which is on the ordinary tank system with a flat bottom. The tank itself—54 m. diameter and 10 m. high—is of iron, but it rests on a bed of reinforced concrete 0·25 m. thick, having a pile foundation. This bed is provided with a surrounding wall 1·65 m. high of the same material, and the space between this wall is filled up with sand. The sand has been covered with a layer of coke breeze mixed with tar, on which the iron tank rests.

The principal reinforced concrete building is the coke house, which has been executed entirely in this material. *Figs. 2, 3, and 4* are various illustrations of this structure, whilst *Fig. 5* shows the building during construction.

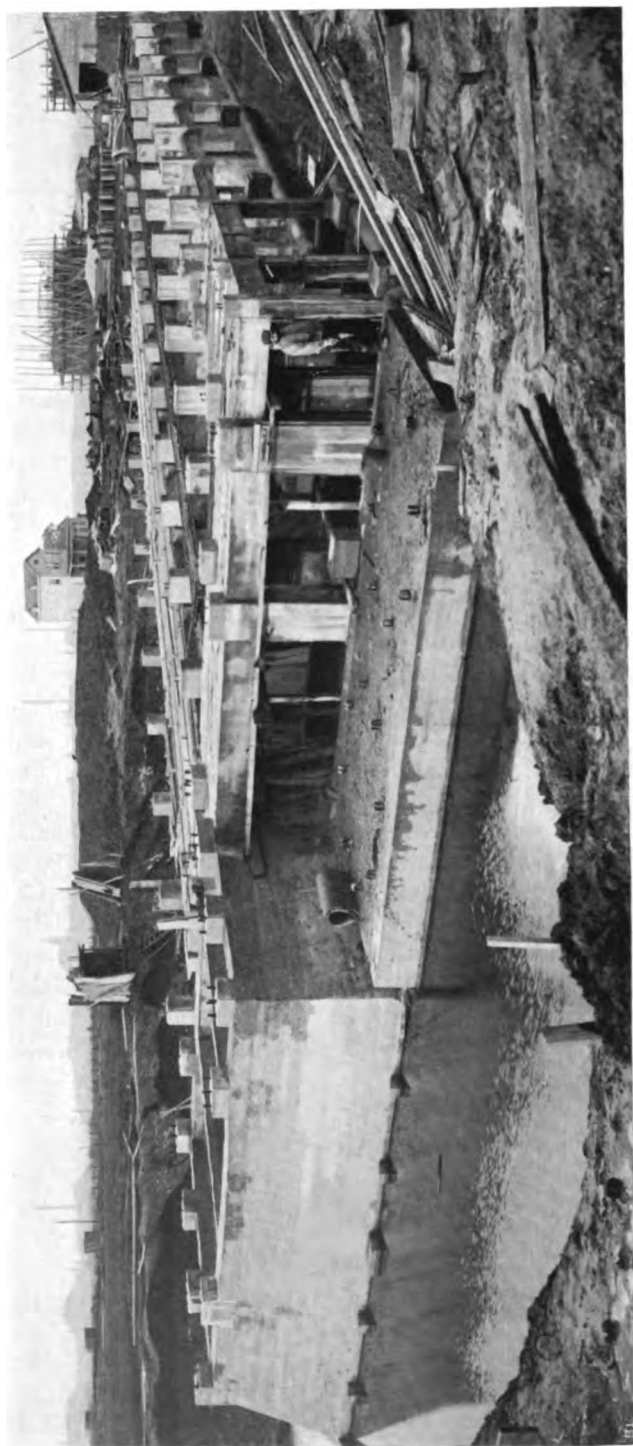


Fig. 1. Reinforced Concrete Foundations to Retort House and Coal Bunker House Annex.
REINFORCED CONCRETE AT THE NEW KEILEHAVEN GAS WORKS, ROTTERDAM.

In Fig. 4, on the right, the retort house is shown with its horizontal furnace chambers. The hot coke is discharged from the chamber ovens and falls into open trucks, consisting of a framework of heavy iron bars. These trucks are conveyed by means of a locomotive to the coke house (Figs. 2



Fig. 2. Reinforced Concrete Coke House.

REINFORCED CONCRETE AT THE NEW KEILEHAVEN GAS WORKS, ROTTERDAM.

and 6). The capacity of these trucks is 8 cu. m., or 3,000 kg. by weight. The truck on arriving at the coke house is placed on to a platform and then lowered into a concrete trough, where the coke is quenched with water. The resulting steam escapes by a high concrete shaft Fig. 4 (a) and the funnel (b) connected with same.

As soon as the coke is quenched the truck is again placed on the moving platform, and this, when raised to its highest point, is tipped with the truck into the chamber marked (e), *Figs. 2 and 3*, from whence the coke discharges into the bunker (d), *Fig. 4*.

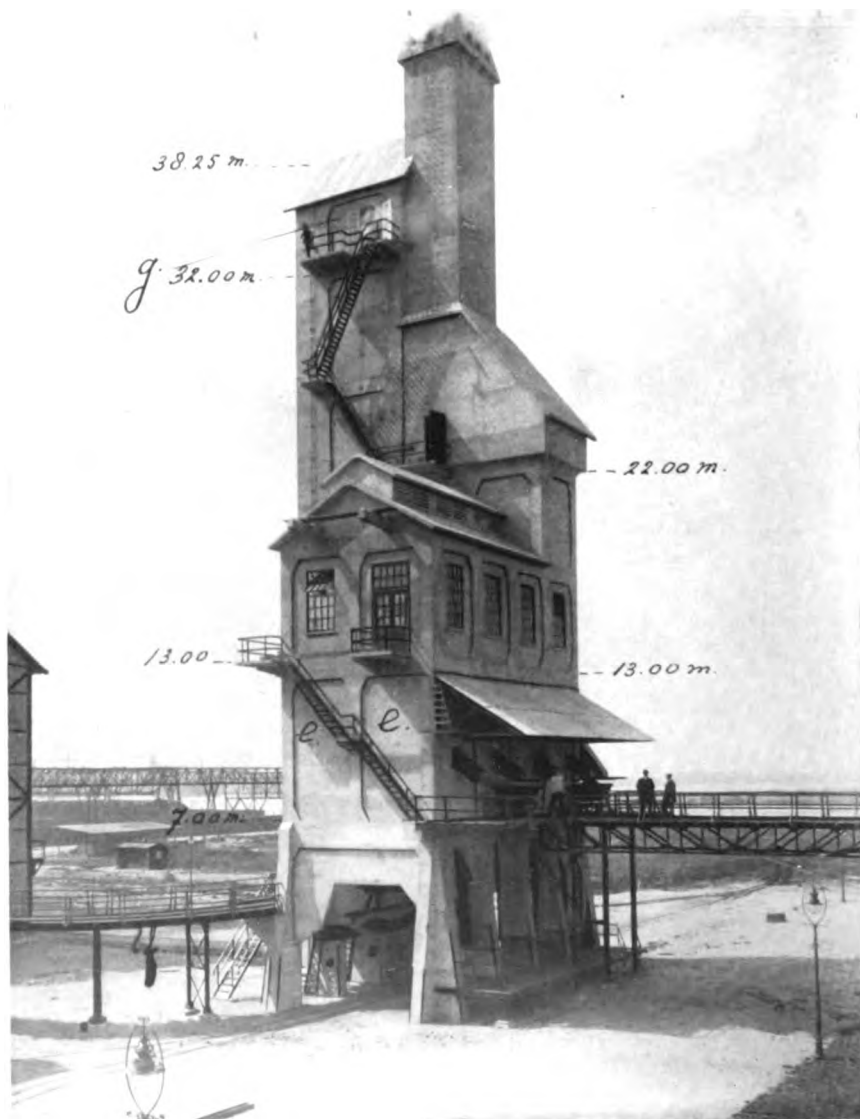


FIG. 3. Reinforced Concrete Coke House with Shaft.
REINFORCED CONCRETE AT THE NEW KEILEHAVEN GAS WORKS, ROTTERDAM.

This building also contains the coke breaking and screening arrangements, and there are four bins (or storage bunkers), each containing coke of different

sizes. From these bunkers the coke can be discharged in various ways, one of which is by means of tip wagons moving on a platform (*Fig. 3*) in connection with the bridges shown on *Fig. 4*, from whence the wagons are emptied from a height of 7 m. There are also devices for discharging the coke into bags

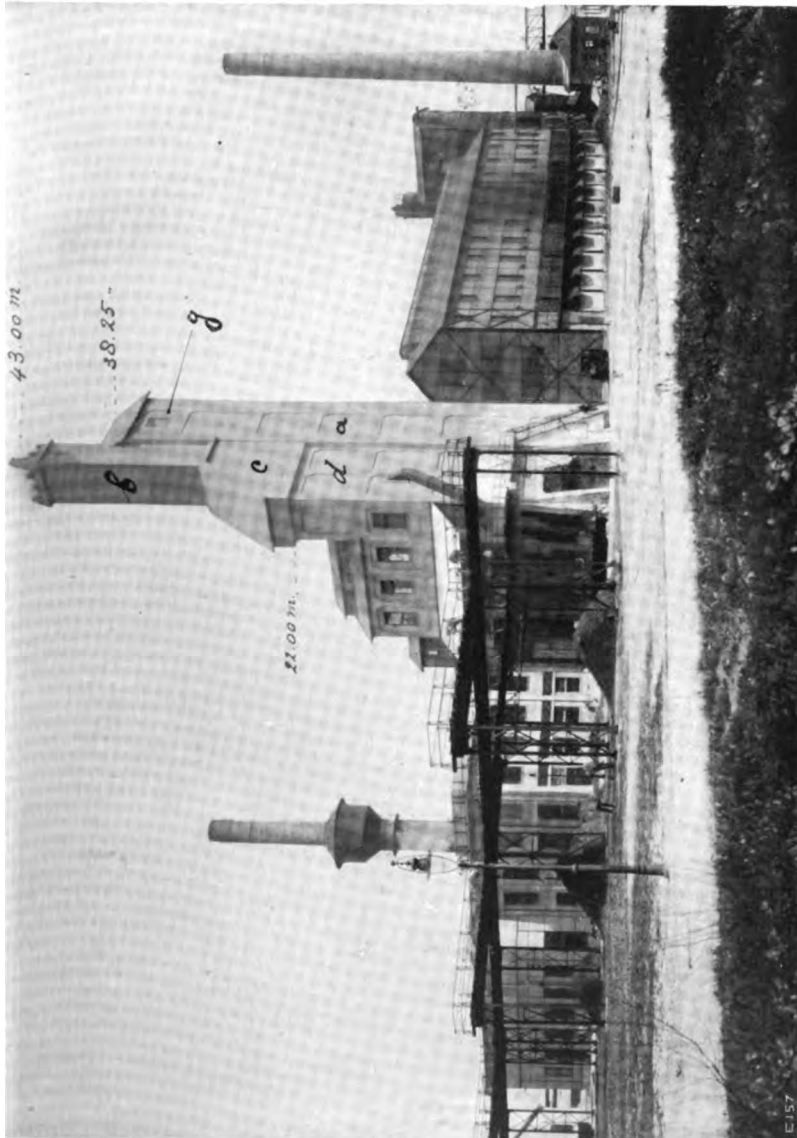


Fig. 4. Coke House with Concrete Shaft and Ovenhouse, and showing in the background the building for the Water-gas plant.
REINFORCED CONCRETE AT THE NEW KEILHAVEN GAS WORKS, ROTTERDAM.

for sale in town, into railway wagons placed under the gate, and into small trucks for the supply of the water-gas plant.

Some other particulars follow here :—

The dimensions of the building are 12 by 6.20 m.

Contents of bunker (d) = 72 m³.

Contents of the four bunkers (e) = 456 m³.

Total weight of lift cage, wagon, and coke $\pm$ 12,000 kg.

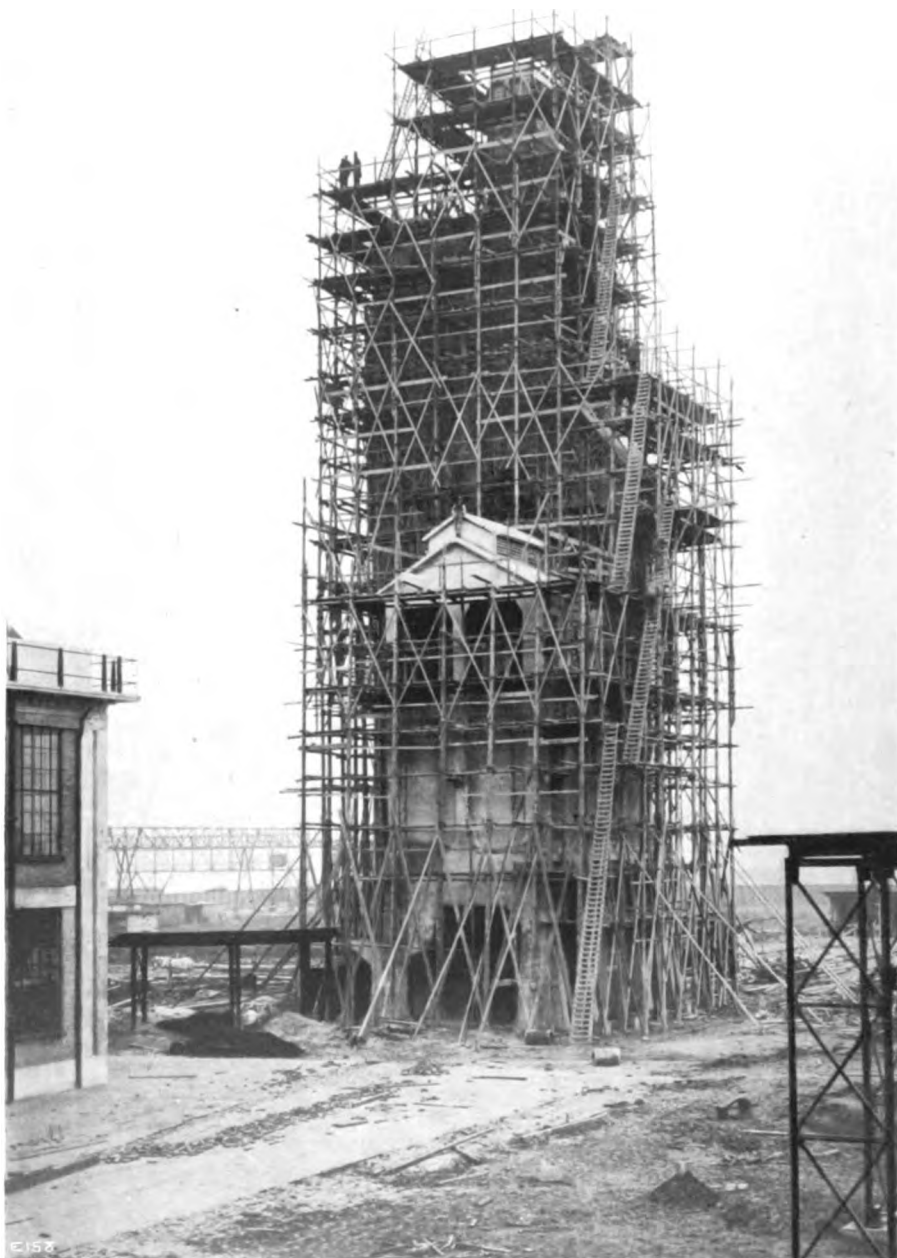


Fig. 5. Coke House in course of construction.
REINFORCED CONCRETE AT THE NEW KEILHAVEN GAS WORKS, ROTTERDAM.

The lift engine is erected above the lift shaft in the engine-room (*g*). The hoisting-motor for the lift has a capacity of 30 h.p. The weight of the lift cage and of the coke wagon is balanced by heavy counter-weights, which move in the shaft (*h*).

The inclined bottom-planes of the bunkers (*e*) are sheathed out with hard tiles, which can be renewed after being worn.

The bottom-planes of the bunker (*d*), in which the unbroken coke is discharged, are sheathed out with iron plates.

The capacity of each break- and screen-apparatus is 34 m³. coke per hour. The building expenses have amounted to:—

± hfl. 41'000 for the building (excluding foundation).

± hfl. 35'000 for the whole mechanical arrangement.



Fig. 6. Showing Discharging of Coke from the Chamber Ovens into Open Trucks
REINFORCED CONCRETE AT THE NEW KEILEHAVEN GAS WORKS, ROTTERDAM.

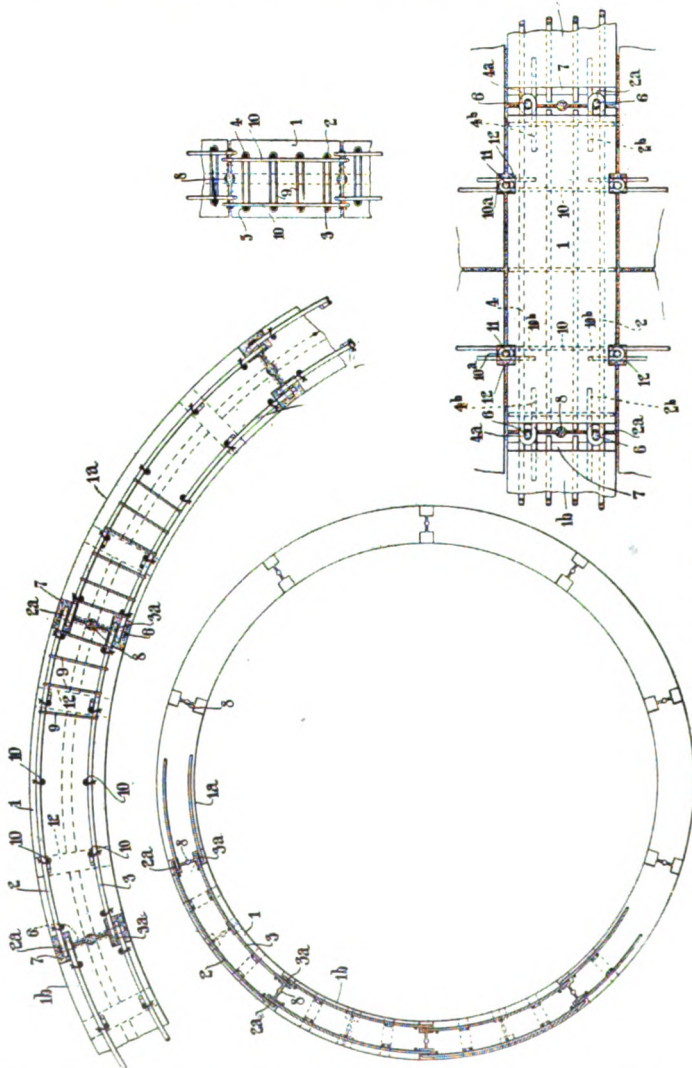
RECENT BRITISH PATENTS RELATING TO CONCRETE.

We propose to present at intervals particulars of British Patents issued in connection with concrete and reinforced concrete. The last article appeared in our issue of April, 1915.—ED.

Culverts, Tunnels, etc.—No. 7120/14. *Edmond Coignet, Ltd., 20, Victoria Street, London. Accepted April 22/15.*—The invention comprises improved reinforced concrete linings, such as described in Specification No. 12,970 of 1911, and while particularly suitable for lining a mine gallery, culvert or tunnel, can be used with equal advantage for constructing a tube or tunnel in place of ordinary circular metal or concrete or similar tubes at present employed for ordinary purposes.

According to the invention a number of longitudinal and transverse independent reinforcing bars, bent over towards their ends so as to form hooks, are employed, the return portions of the latter being embedded in the concrete, leaving projecting loops or eyes; there are pockets at the ends and sides of the concrete for the reception of the looped ends of the reinforcing bars.

The segmental numbers (1) are made of a width and length that will enable them to be conveniently handled and manipulated, and of a form that can be readily moulded; upon their meeting faces keying spaces are provided into which the projecting hooks formed by the main reinforcing members enter, the projecting hooks being coupled together by means of metal bolts, etc.



FIGS. 1, 2, 3 & 4. CULVERTS, TUNNELS, ETC.

The main reinforcing bars 2, 3, 4, 5, for the unit 1, are bent round at the outer ends to form hook-like loops (2a, 3a, etc.), the return ends (2b, etc.) being embedded in the concrete. Through the projecting loops thus formed, gudgeon or uniting pins or bolts (6) may be passed, leaving the pockets or openings (7) free for the cement grouting to be run freely therein and to pass through the spaces (8) so as entirely to fill and close the spaces between each pair of unit members, such as 1, 1a, 1b, thereby forming not only a complete joint but a means for holding in position the gudgeon or dowell pins or bolts (6), the outer surface of the pockets being finished to suit the configuration of the inner and outer edges or faces of the unit members.

The bars 2, 3, 4, 5, are held in position by means of stirrup members (9) and cross-bars (10), uniting these to the main members by binding wire or other ordinary means of attachment to retain them in position until the concrete or cement surrounds them when poured into the casing or mould in which each unit is constructed.

Longitudinal connections are made by means of the loops 2a, 3a, etc., and the cross connections by means of similar loops (10a) placing through these latter loops connecting pins (11), to hold them in position, leaving pockets and spaces (12) for the cement grouting to be run therein so as to bond the different segments into position and to build up the different sections of the tube to make the length of circular gallery or tunnel desired.

The meeting ends of the different units are arranged so that the joints are broken or alternated in the manner shown in Fig. 3.

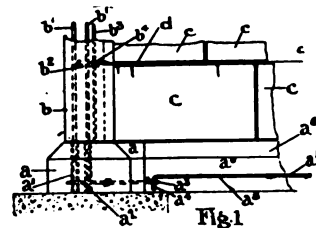
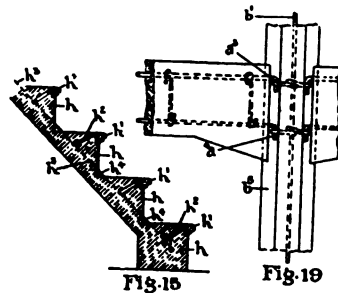
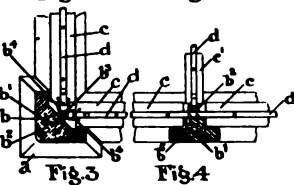
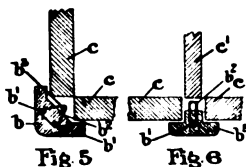
Reinforced Concrete Buildings.—No. 10639. C. M. Burnett, St. Catharine's, Clifton Road, Regent's Park, Shirley, Southampton. Accepted March 18/15.—In

accordance with this invention buildings are formed with a reinforced concrete framework consisting principally of matured reinforced concrete beams, columns, slabs, etc., provided with projecting reinforcement by means of which the various parts are connected together, the joints being permanently fixed before completion by filling in the space between the concrete ends with Portland or other cement.

A corner foundation base stone (a) carries tubes (a') into which fit the projecting reinforcing bars (b') of the corner column (b). These tubes (a') are embraced by a metal bar (a'') which has projecting eyes (a3) into which are fitted the turned-down ends (a4) of reinforcing bars (a5) for the foundation beams. Breeze or like slabs (c), and a partition slab (c') at an intermediate column (b5), Figs. 4 and 6, are fitted between the columns, and cross bands or wires (d), secured to the blocks, are provided with projecting down-turned ends which hook into loops (b2) projecting from the columns. In the case of the corner column (b) a loose reinforcing bar (b3) may be carried by the stirrups (b2), and the strips (d) are hooked into loops (b4) passed round the rod.

Fig. 19 shows the manner in which a matured reinforced concrete floor beam is connected to an intermediate column with projecting portions (b5) forming a T-section as in Fig. 4; the ends (a4) of the beam-reinforcing bars are turned down and hook into eyes cast in the column and passed round the reinforcement.

After the various units have been assembled in this manner they are rendered monolithic by casting cement between them.



REINFORCED CONCRETE BUILDINGS.

Fig. 15 shows a reinforced concrete stairway, provided with wood or concrete nosings (h_1) attached by metal bands (h_2) to a longitudinal reinforcement bar (h_3), which is exposed at the angle of each step to form an eyelet for the stair rod.

Joints are made in the columns by metal sockets surrounding the reinforcement; they project from the surface of one part and engage corresponding recesses in the other.

Moulds for Walls.—No. 9193/14. W. J. Mills, Church Road, Whitechurch, Somerset. Accepted April 8/14.—This invention provides an improved form of mould for casting hollow walls *in situ*, the feature of the invention being the cam plate catch for connecting the shutters to the cavity block.

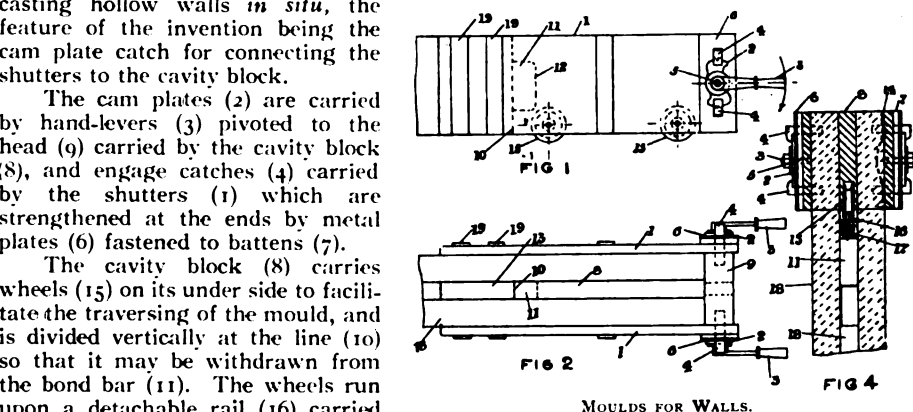
The cam plates (2) are carried by hand-levers (3) pivoted to the head (9) carried by the cavity block (8), and engage catches (4) carried by the shutters (1) which are strengthened at the ends by metal plates (6) fastened to battens (7).

The cavity block (8) carries wheels (15) on its under side to facilitate the traversing of the mould, and is divided vertically at the line (10) so that it may be withdrawn from the bond bar (11). The wheels run upon a detachable rail (16) carried by a wooden bar (17) which rests upon the bond bars (11) in the cavity of the wall.

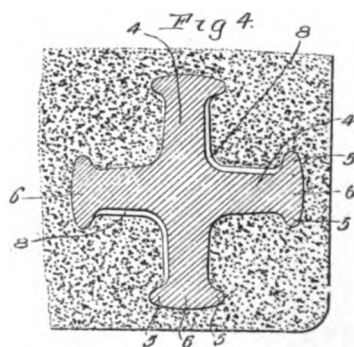
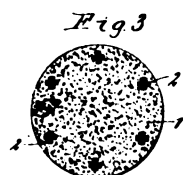
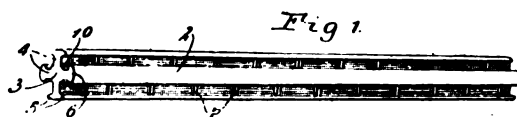
To assist in separating the shutters after the cam plates have been turned to release the catches, springs (14) are provided between the shutters and the head (9). A wooden clamp is provided to fit over the shutters of the mould between battens (19), and both shutters and cavity blocks are covered with a protective layer of sheet zinc, which serves to form a smooth wall surface.

Reinforcing Bars.—No. 10353/14. Patrick Henry Kane, 63, Cleveland Avenue, Buffalo, Erie, New York, U.S.A. Accepted February 11/15.—The present invention consists essentially in a reinforcing bar of cruciform cross section for use in beams, columns and other forms of reinforced concrete work.

Each bar may be provided with anchoring means in the form of projecting ribs or



MOULDS FOR WALLS.



REINFORCING BARS.

corrugations, or grooves may be provided so as to effectively prevent any slipping or displacement in the concrete.

Each of these bars consists of a core or body (3) and four or more longitudinal webs (4) extending outwardly from the body in different directions and provided with laterally-projecting anchoring heads, flanges or enlargements (5) which extend beyond both sides of each web.

Adjacent webs of the bar with their laterally-projecting flanges form an undercut space which forms a bond between the bar and the concrete.

To anchor further the reinforcing bars in the concrete structure, and to prevent slip, they may be provided with projecting ribs or corrugations (7) of any suitable form, as shown in *Fig. 1*; or, if desired, anchoring grooves (8) may be substituted for these ribs, as shown in *Fig. 4*; or both the ribs and the grooves may be employed.

The method in which the bars are manufactured by rolling is also described.

Reinforced Concrete Floors.— No. 17561/14. W. G. Shipwright, 218, Well-meadow Road, Catford, London, S.E., and F. R. Martin. Accepted March 4/15.—This invention comprises an improved floor of the type built up of beams made of burnt clay or like blocks and reinforced concrete without employing any temporary supports.

The bricks or tiles are approximately inverted V or arched cross-section, so that when a number of these tiles are arranged end to end and connected by means of reinforcing rods embedded in cement in longitudinal holes or grooves in the tiles to form a beam or slab, and when two of these beams or slabs are placed side by side in contact with each other an approximately V-shaped space is formed between the beams.

The tiles (A) have sloping or arched sides, and dovetail grooves (a) are formed in the sides and bases. A number of these tiles are placed end to end, resting upon their flat bases, and connected to each other to form a beam or slab by embedding metal rods (B), or bars or angle irons, in concrete or similar material introduced into grooves (B) formed in the sloping or arched sides and at the apex of the tiles. The beams are ready for use as soon as the concrete has set hard in these grooves (B). The tiles may be hollow as indicated in dotted lines in the drawing.

In a modified construction, *Fig. 6*, the tile beams are spaced some distance apart, and shallow tiles (C), shaped about as shown in the drawing, are arranged in the bottom of the approximately trough-shaped space between adjacent beams before the space is filled in with concrete, the tiles resting upon the sloping sides of the beams.

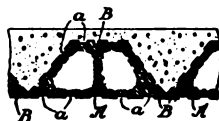


FIG 4

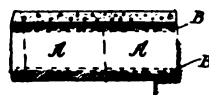


FIG 5

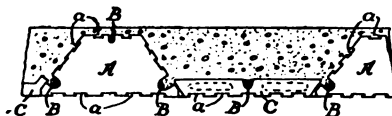
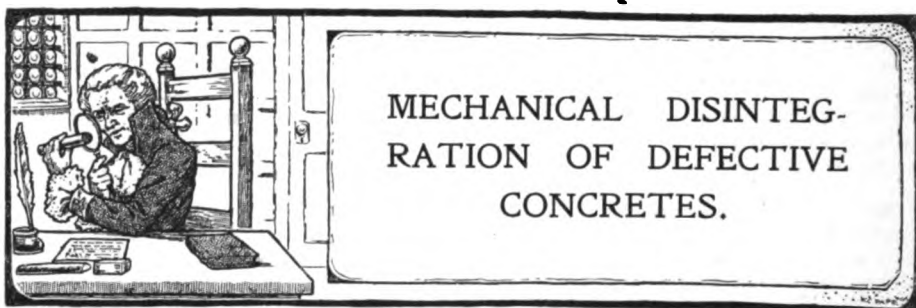


FIG 6

REINFORCED CONCRETE FLOORS.



**Demonstrating Certain Disruptive Crystalline Actions,
which have their inception in the Capillary Pores.**

By NATHAN C. JOHNSON,
Consulting Engineer, New York City.

Second article of series.—In reprinting these articles we would state that the opinions expressed are those of the author, and the Journal as such does not necessarily associate itself with the conclusions arrived at. The articles are reprinted by the courtesy of the "Engineering Record," U.S.A., and the illustrations have been placed at our disposal by the author.—ED.

WHEN sea water, or even fresh water, is brought into contact with the cement matrix of concrete, solution to a greater or less extent adds new compounds of indefinite nature. When such waters are carried to the interior of a mass of concrete by the capillary action of minute pores, the limitations to quantity imposed by the size of the pores result in the production of super-saturated solutions of these dissolved materials. But super-saturated solutions speedily crystallise on reaching a certain density, with the result that there are exerted on the matrix of the concrete powerful disruptive stresses, the effects of which may readily be traced by means of the microscope. These stresses may even exceed the strength of the concrete, with the production of local disruptions which later become general. To differentiate these disruptions from other purely chemical actions that may cause the failure of concretes, they have been designated as "mechanical disintegrations"—a designation which, it is believed, the demonstrations of this paper will justify. These are some of the secondary effects attributable directly to the presence of entrained air in concrete, the primary effects of which were pointed out in the preceding article (issue of June).

It should be understood that while these demonstrations are unusual and to a certain extent startling, they relate to defective concretes, and are quite without prejudice to the successful uses of concrete in sub-aqueous structures in all parts of the world. One of the primary objects of this research has been the production of concrete thoroughly capable of withstanding the action of waters which have heretofore caused trouble; and in any such quest it is essential that all of the actions involved should be thoroughly understood.

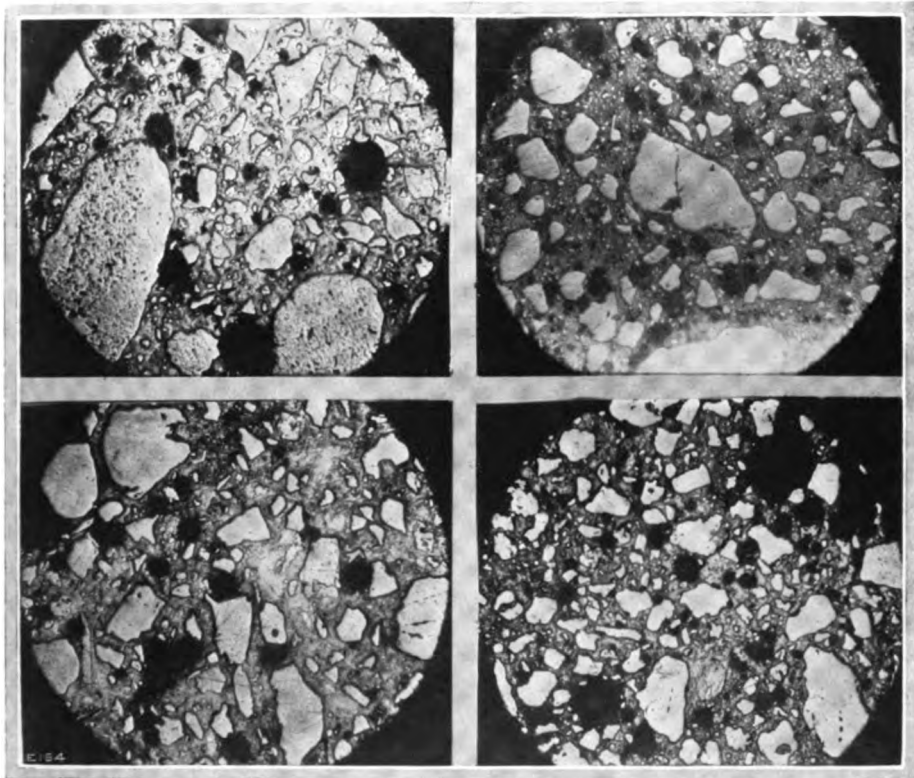
It is probable that many of the vexatious faults of concrete construction of all classes are due as much to physical as to chemical causes; and, secondarily, to a combination of physical and chemical causes. In sea-water construction, especially, it is probable that the defects which induce failure lie primarily not so much in the cement, providing cement of proper quality be used, as in the improper proportioning of aggregates, or in poor mixing, or in a combination of the two.

CONCRETES TO BE STUDIED.

In Fig. 7 are given photomicrographs of the four concretes which are to be investigated with a view to determining causes of such failure. These photographs were taken at low power in order that as much of the surface as possible, the inter-relation

of aggregates, and, particularly, the prevalence of pores in the matrix might be made plain.

In interpreting these photographs, it should be remembered that all of the larger irregular white patches with dark borders are sand grains; that each of the black patches, or irregular black streaks, is a hole or a deep crack, and that the white material between the sand grains is the matrix, of more or less hydrated cement. Is it not significant that no two of these sand grains are touching? And is it not significant also and confirmatory of what has before been said, that between the sand grains in every photograph lie smaller grains, too small to be sand? By using a higher magnification with the microscope, these latter can very easily be proved to be groups of unhydrated



(a) Steel Pier, Atlantic City, N.J.
(c) Dock Wall, East River, New York.

(b) Spillway of Dam, Ithaca, N.Y.
(d) Pile Cap, New York Harbour.

Fig. 7. Investigation of Concretes discussed in this article.
Magnification, 20 diameters; black spots are holes.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

cement particles. a proof to be demonstrated fully and conclusively in the final article of this series.

PORES IN MATRIX.

But the feature to which present attention is most strongly directed is the prevalence of black spots in the cement matrix. Each of these is a hole, or pore, of indefinite direction and extent. Further, as each photograph shows the features of a random section in a random specimen from the structure in question, it is probable that practically the whole of each of these structures is literally honeycombed by millions of tiny pores.

Although the presence of such pores in concrete has long been known, it is a question whether or not their prevalence and the damage they may cause has been recognised. The seepage of water into pores and the disintegrating action of frost have been remarked by a number of observers, but, so far as the writer is aware, the really serious extent to which such actions may operate has not been appreciated, the inferential assumption, at least, being that they are confined to surface layers. On the contrary, however, it seems probable that these actions operate beyond the surface layers, to a depth dependent upon the degree of porosity of the mass.

Such a statement requires rigid proof. As a first step toward it, consider what must be the state of affairs in a concrete as full of tiny pores as the concretes under examination would seem to be.

CAPILLARY ACTION.

The phenomena of capillarity—exhibited, for instance, by a tube of hair-like diameter of bore, or by a lamp wick—are well known. The smaller the bore of the tube, or the closer the weave of the wick, within certain limits, the greater will be the capillary effect and the higher will the liquid rise.

It is probable that the pores of the concretes under examination are all within the capillary range and well adapted to aid the entrance of liquids in the mass. Further, the phenomena of *absorption* and *adsorption* may aid such entrance, transferring the liquid through semi-solid barriers to other capillary passages beyond until the entire mass has become thoroughly permeated.

Under such conditions a number of disintegrating actions, in addition to that of frost, are possible. To take one of the most obvious, consider the results of contact between a strongly saline, sulphated water and steel reinforcement embedded in concrete. Of all natural waters, saline solutions are the most corrosive of iron and steel; and because of the progressive nature of such corrosion, a single point of attack soon results in the affection of a large area of the material. Ferrous sulphate and ferric chloride are formed, parts of which, in combination with the oxygen entrained in the concrete, or in the entrant water, break down into $\text{Fe}_2\text{O}_3 \times \text{H}_2\text{O}$, or common rust, with an enormous increase in bulk. Destruction of reinforcing steel is a sufficiently great evil, but when there is added to it the actual disruption of the concrete by which it is surrounded, collapse of the structure is to be expected. However, the existence of such action is dependent upon the continuity of the capillary pore and upon the existence of a point, or of points of attack, on the surface of the steel. Extended experiments along these lines show that where proper contact is had between the cement and steel there is little liability to corrosion.

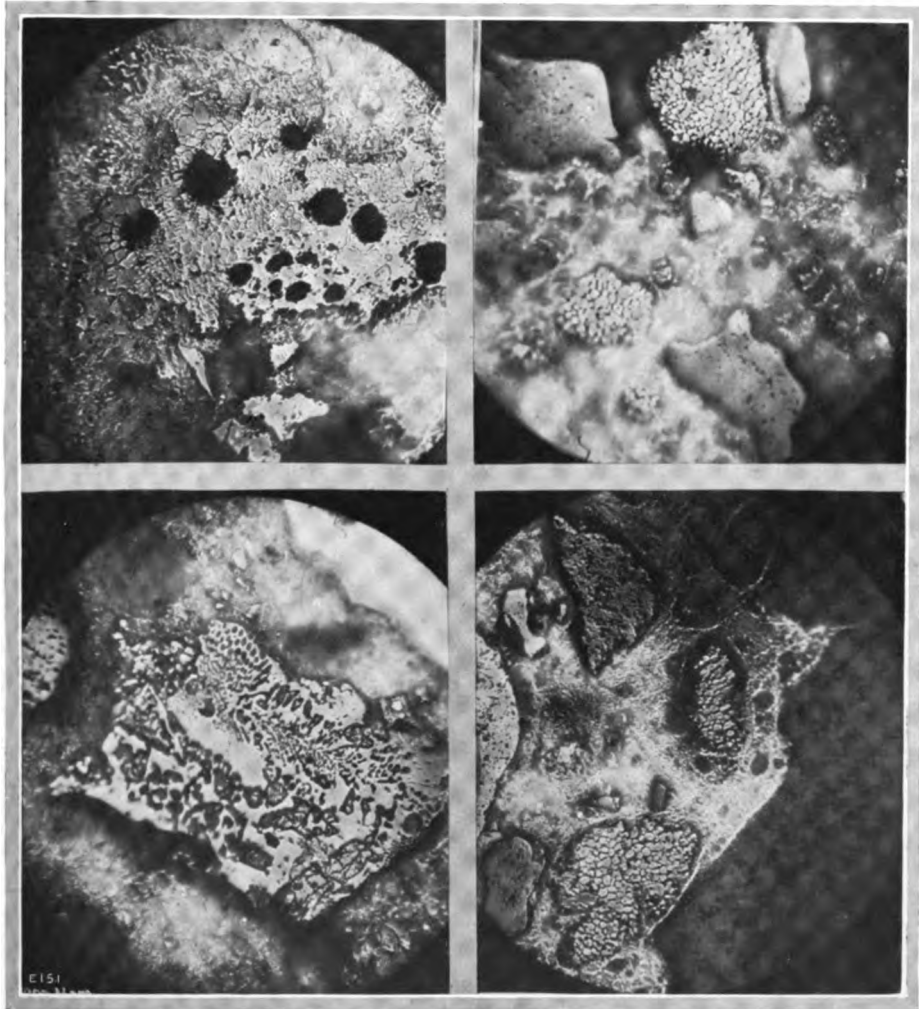
UNHYDRATED CEMENT.

But although frost action and corrosion are often important factors in the success or failure of under-water concretes, other more obscure physico-chemical actions are properly the subject of this present investigation. As in previous papers, the microscope will be used to furnish visual evidence in regard to doubtful or conjectural points. Abstruse chemical questions will be avoided, as far as possible, it being preferred herein to regard cement only as a physical material available for structural purposes, rather than as a chemical complex capable of giving certain reactions under specified conditions. Such problems as the latter are the province of the chemist, rather than of the engineer.

It was stated in the previous article that all concretes, regardless of age, contain a considerable percentage of unhydrated cement particles, usually in groups or lumps. Such groups are shown in *Fig. 8*, high-power photomicrographs of the four concretes whose general characteristics were shown in the low-power photographs of *Fig. 7*.

It will be seen that all the photographs exhibit the same characteristic group-

structures. Especial attention is called to (c), *Fig. 8*, not only as an unusual photograph, but also because it shows a single particle of what is apparently cement clinker eutectic. Needless to say, this particle has remained quite unaffected by water, despite the passage of years, although the concrete from which was taken the sample in which this relatively large piece lies was placed under water 34 years ago, when fine-grinding



(a) Spillway of Dam, Ithaca, N.Y.
(c) Dock Wall, East River, New York.

(b) Pier at Atlantic City, N.J.
(d) Pile Cap, New York Harbour.

Fig. 8. Unhydrated Cement Particles; magnification, 200 diameters.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

was almost unheard of. This of itself is a very significant comment on the relative values of fine and coarse grinding. This is the only concrete so far examined by the writer in which so large a piece of clinker has been observed.

PART PLAYED BY UNHYDRATED CEMENT.

It now becomes necessary to conjecture the part such lumps play in the matrix of partially hydrated or wholly hydrated and set cement lying between the aggregates, if

water is later brought to them by capillary pores. Necessarily, these lumps form spots of low mechanical strength, but it is possible in a defective concrete that such may be their least objectionable feature, for in that capacity they at least form an integral part of the rigid mass. But if hydration is brought about by the means above outlined, a new set of reactions is necessarily set up, with the formation in the very heart of the matrix of new compounds of uncertain nature and larger space-filling value. (H. S.

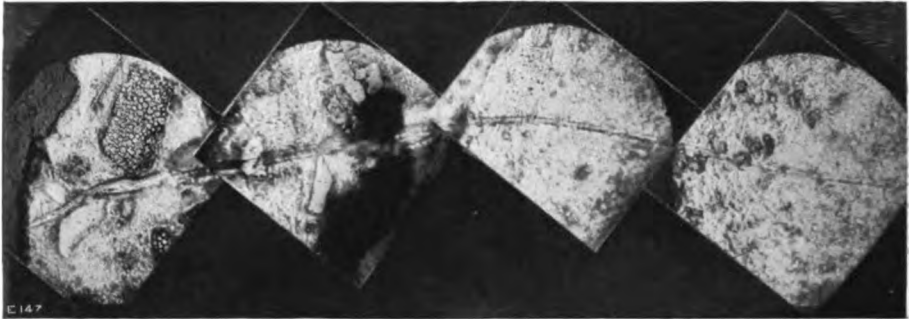


Fig. 9. "Shear Planes" or "Strain Bands," Pile Cap, New York Harbour; magnification, 200 diameters.

Spackman, *Cement Age*, August, 1908; Feret, "Annales des Ponts et Chausses," 1892, II. p. 93.) But if this action takes place, and if compounds are formed such that tremendous disruptive forces are exerted on the matrix, it may reasonably be expected that their effects can be detected even short of actual fracture. In the softer metals, such as babbitts, which in texture are not unlike the cement matrix of concrete,

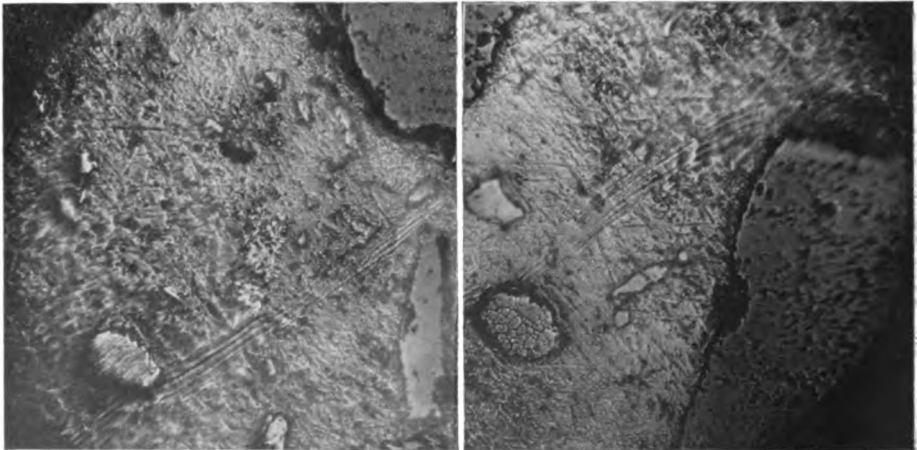


Fig. 10. Further Examples of "Shear Planes" or "Strain Bands."
THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

incipient failure leaves traces known as "shear planes," or "strain bands," which are very distinctive. In Fig. 9 are shown such "shear planes" in the cement matrix, with several unhydrated cement groups near by, in a proximity highly suggestive, if not wholly convincing. The pore in this instance may be the fissure at the side of the sand grain at the left. Fig. 10 shows two other such shear planes in another specimen.

ACTION OF SALTS CARRIED BY SEA WATER.

This suggestion as to the origin of these shear planes is made tentatively. Their existence is known beyond doubt, but the nature or causes of the forces which gave rise to them is solely a matter of conjecture. There are other physico-chemical actions, not attributable to the hydration of lumps of cement, which might, with equal probability, be responsible

For instance, when a tiny pore has sucked up its fill of sea water, what becomes of that water? It can hardly be expected to remain *in situ* as so much sea water, until the end of time, for it is loaded with chemically-vigorous salts. Absorption and adsorption must affect it, both qualitatively and quantitatively. First of all, it dissolves as much

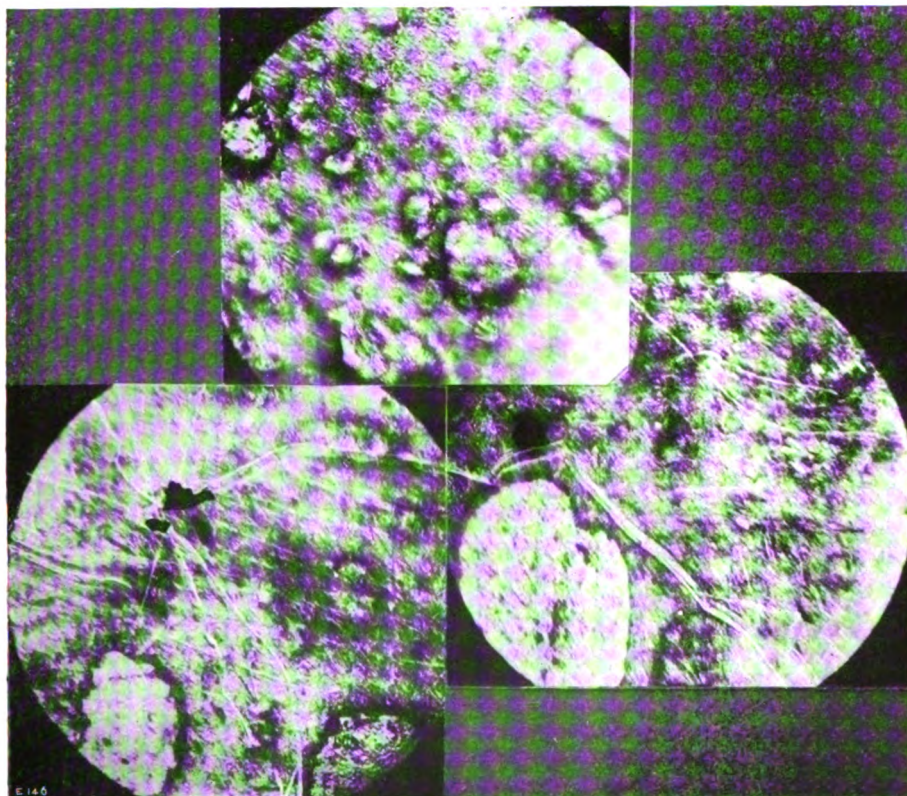


Fig. 11. Matrix Cracks radiating from a pore: Pier at Atlantic City.
Magnification, 160 diameters.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

as possible of the matter with which it is in contact. When saturated, it deposits in solid form, and possibly with increased bulk, the salts it holds in solution, generally with the result that the concrete in the vicinity is at least softened, if not wholly disintegrated. This softening is well known and has been often commented upon. (Taylor and Thompson, "Concrete, Plain and Reinforced," page 310; H. S. Taft, *Cement and Engineering News*, May, 1914, page 126; U.S. Bureau of Standards, Bulletin 12, page 100.)

In the progress of such an action, it is reasonable to expect that a pore would be the starting point of any disintegration. Such an expectation is justified. In Fig. 11 is shown a plane cut across a pore (indicated by the black spot); and radiating from this

D

pore are *no less than six fractures*, with probably others in different planes which are not seen. The fractures can readily be traced because of their aimless wanderings, closely resembling creeping-cracks in a sheet of glass. It should be noted that these fractures are quite distinct from the polish scratches, which go straight across the photograph.

CRACKS REPRESENTATIVE, NOT UNUSUAL.



Fig. 12 (a). Front and rear of Defective Buttress Wall at Pittson, Pa., being disrupted by crystalline dikes.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

seepage show these features to a marked degree. Magnify the actions indicated in Fig. 11 a few hundred times, and there is realised a condition of actual, visible disintegration similar to that shown in (a) Fig. 12, where the tiny fracture has become a large crack filled with crystalline matter that sticks out a quarter of an inch or more from the face of the wall. And in (b) of the same figure is made evident the cause of this disintegration. As can clearly be seen, this wall is but a shell of poorly compacted and almost uncemented stones, into which the ground water has the freest of access. No wonder that the grouted face shows evidence of the decayed body. Each failure results from determinable causes; and this case is but further evidence that in most instances the causes have their origin in the wilful abuse of good materials.

It is quite possible with the data already in hand, and with more accumulating daily, to multiply similar instances of incipient disintegration to a very considerable length, but two cases more should here suffice to bring home to engineers a realisation of the actual existence and potency of crystalline action similar to those noted above.



Fig. 12 (b). Front and rear of Defective Buttress Wall at Pittson, Pa., being disrupted by crystalline dikes.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

PROGRESS OF CRYSTALLISATION.

In Fig. 13 is shown a long crystallising deposit, or dike, radiating from a pore. The process of its formation is not hard to picture. In the beginning the pore sucked up the sea water, loaded with its various salts. In the manner before out-



Fig. 13. Disruptive Crystalline Dike in Concrete Pier, Atlantic City, runs through matrix and around aggregate: magnification, about 60 diameters. THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

lined, the water content was diminished, with resultant concentration of solution, until its dissolved salts crystallised, possibly while the concrete was yet comparatively new. Room had to be made for these crystals, so that a tiny fissure in the weakest material, i.e., the cement matrix, was formed. But surfaces close together, as were the sides of this fissure, act as do capillary pores, so that a fresh supply of water brought by the pore was sucked into this fissure, where its salts, in turn, crystallised, wedging the fissure yet further along and wider apart, with the formation, by repeated crystallisation, of a hard dike of foreign material in the space thus provided. This process was repeated again and again, an indefinite number of times, until the supply of crystallising salts was stopped in July last by the removal of the specimen from a beam beneath a pier at Atlantic City.

This dike, like all others of its kind, meanders through the matrix, *never crossing a piece of aggregate, but keeping to the weakest part of the concrete.* At the end near the pore, where the crack began and where the wedging was most often repeated, the dike is the widest. This portion shows black in the photograph, indicating a surface out of the focus of the microscope. The dike itself at this place and the crystals in the pore have either been torn out in polishing or have been dissolved out by too great supplies of fresh water, or they may have become loosened by drying and fallen out. In any event, they are gone, but the place where the dike lay is plainly seen. The rest of the crystalline filling stands out boldly, because it is stronger and harder than the matrix which it has crushed.

As stated before, this action is not necessarily confined to defective sea-water concretes. Fig. 14 shows two similar radiating dikes, with the beginning of a third, in a fresh water concrete of like nature. This latter dike may have stopped in the early stages, as the other two may have required all of the dissolved matter that the pore could furnish, or it may have dropped beneath the surface polished, or have risen above and been cut away in the grinding of the specimen. In any event, it has disappeared. It will be observed that these dikes meander as did the others, dodging the strong aggregate and keeping to the weak matrix. These conditions are probably duplicated a few million times in the structure from which

this sample was obtained, as it is even now showing outward signs of the inward disintegration of which this is a type.

SUMMARY.

Summarising this article, it should be noted:—

1. That concrete structures in which proper care has not been exercised in selecting aggregates, or in mixing and placing, are honeycombed by millions of pores of greater or less size, formed by the occlusion of air in the cement matrix.

2. That these pores are adapted to promote by capillary action the penetration of water below the surface layers.

3. That when water is brought by a capillary pore, aided by absorption, or adsorption, or both, into contact with a group or groups of the unhydrated cement particles that exist in all concrete, new reactions with new volumes are probably set up in the rigid matrix of set cement.

4. That these reactions may cause disruptive stress to be

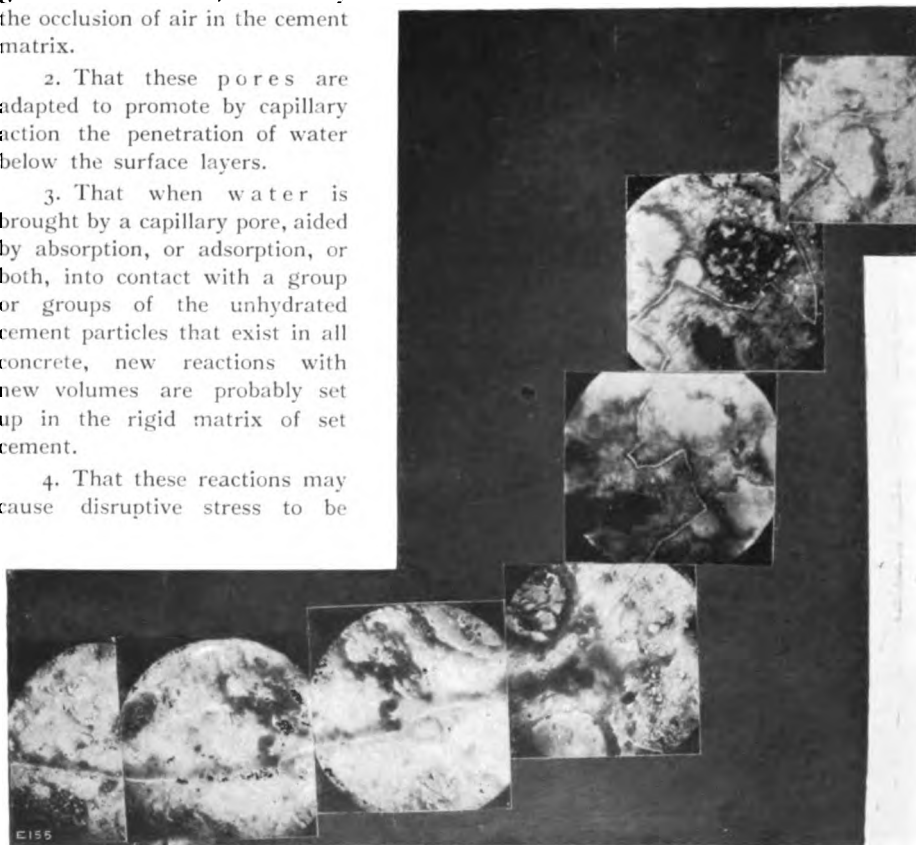


Fig. 14. Disruptive Crystalline Dike, Spillway of Dam, Ithaca, N.Y.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

exerted on the cement matrix of the concrete, producing "shear planes" or "slip bands" as evidence of the local failure thus produced.

5. That "shear planes" and other evidences of local failure by disruption, whether resulting from the hydration of cement or from other causes, have their origin in a minute pore or fissure. *

6. That other disintegrating actions are produced by water entering through pores into the mass, by reason of this water, whether fresh or loaded with chemically-active salts, dissolving portions of the cement matrix.

7. That when such entering waters are sufficiently saturated with soluble compounds, deposition in crystalline form takes place, with tremendous disruptive forces exerted on the surrounding material, i.e., the cement matrix.

8. That these disruptive forces pressing on the walls of the confining pore produce one or more fissures in the matrix, and that these fissures, in turn, act as capillary passages, with repetition of seepage, crystallisation, and wedging, the crystalline water filling in as the passages lengthen and widen, until a group of hard, disruptive progressing dikes effect actual disruption.

9. That this action is not confined to concretes exposed to sea-water, but affects other concretes, such as fresh-water dams, filter-beds, buttress walls, sidewalks, etc., where defective materials or construction permit the entrance of water into the mass.

10. That the responsibility for concrete failures of many kinds need not necessarily rest solely upon the cement, but that other factors, such as proper choice of aggregates, efficient mixing, proper consistency, and care in placing must be held equally responsible, since through these latter come the inducing causes of failure.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

THE CONCRETE INSTITUTE. •

THE STABILITY OF QUAY WALLS ON EARTH FOUNDATIONS.

By F. E. WENTWORTH-SHEILDS, M.Inst.C.E., Vice-Pres. C.I.

The following is an abstract from a Paper read at the Sixty-first Ordinary General Meeting of the Concrete Institute. A discussion followed, of which we also give a short report.

In spite of the large amount of experience which has been gained in the construction of quay walls, it is still one of the most difficult problems in engineering to design a wall on an earth foundation with confidence that it will be stable when completed. Even if the designer of such a wall is assured that it will stand, he cannot with any confidence tell you what factor of safety it possesses. The cause of his uncertainty is, of course, the difficulty of ascertaining the actual lateral pressure imposed by an earth backing and the actual resistance offered by an earth foundation. His difficulties are thus quite different from those of the engineer who has to design large masonry dams. The latter structures are invariably placed on a foundation of solid rock, and the designer's chief care is that the stresses in the masonry of which the dam is composed shall not exceed a safe limit. The dock engineer, on the other hand, has to be anxious that his wall shall not move as a whole on the comparatively soft material on which such structures have in general to be placed.

The object of this paper is to consider the uncertainties and difficulties which the designer of a quay wall has to face, and, if possible, to ascertain how far calculations can assist him, and how far he must trust to judgment based on experience. It is also intended to urge upon this Institute the importance of collecting information on this subject in the hopes that by degrees these difficulties and uncertainties may be cleared away.

A retaining wall may fail as a whole in two ways: (1) By sliding forward on its base, and (2) by overturning. It may be said at once that as regards quay walls, at least, the former mode of failure (by sliding forward) is by far the more common.

Conditions of Stability in a "Gravity" Wall.—The conditions may be summarised thus: The forces tending to thrust the wall outwards (generally the lateral pressure of the earth backing) must be at least equalled by the forces tending to restrain it or thrust it inwards. The latter forces are generally the pressure of the water in front of it, the resistance of the earth in front of its toe, and the horizontal resistance to shear (or the friction) at the base of the wall. If these horizontal forces balance, the wall cannot slide forward.

The resultant of the outward forces, however, is almost always at a higher level than the resultant of the inward forces. Thus a couple is formed tending to overturn the wall about its toe. This couple induces a counter-couple tending to keep it upright. The forces forming this counter-couple consist, on the one hand, of the weight of the wall acting vertically downwards, together with the weight of any earth or water

which may lie above the base of the wall, and, on the other hand, the upward resistance of the earth under that base.

If the upward resistance of the earth beneath the wall is capable of forming with the downward weights a couple at least equal to the overturning couple, the wall cannot overturn.

In order that the earth beneath the wall shall be capable of forming this righting couple, two things are necessary. It is obvious that the centre of the earth's resistance must be forward of the centre of gravity of the wall and of other loads on the base, and generally it is forward also of the mid-point of the base of the wall. Consequently the intensity of upward resistance is generally greatest at the toe and least at the heel. To preserve stability the resistance at the toe must not be greater than the maximum which the earth is capable of offering, and that at the heel must not be less than the pressure induced by the tendency of the earth to rise at this point.

Thus it will be seen that if the stability of any given wall is to be ascertained by calculation, the following forces must be determined:—

(1) The amount and position of resultants of horizontal outward forces, such as lateral pressure of backing, lateral pressure due to surcharge, pull of ships' moorings on bollards.

(2) The amount and position of resultants of horizontal inward forces, such as pressure of water in front of wall, resistance of earth in front of toe, resistance due to friction at base of wall.

Note.—The forces of paragraph (2) must be at least equal to those of paragraph (1).

(3) The overturning moment produced by inward and outward forces.

(4) The amount and position of resultants of vertical downward forces, such as weight of wall, weight of backing resting on wall, weight of water on toe, friction between backing and back of wall, friction between toe and earth in front of it.

(5) Consequent amount and position of resultant of upward vertical forces, such as reaction from earth under base.

Note.—The position of this resultant can be obtained by equating the righting moment produced by the vertical forces to the overturning moment due to the horizontal forces.

(6) The consequent maximum and minimum intensities of reacting pressure of earth under base at toe and at heel respectively.

(7) The maximum permissible reacting pressure at toe, and the minimum permissible pressure at heel.

Note.—The actual pressure at the toe (referred to in paragraph (6)) must not be greater than the permissible pressure (referred to in paragraph (7)), and the actual pressure at the heel must not be less than the permissible pressure.

This method of ascertaining the stability of a wall is illustrated by an actual case in Appendix I. The same results can, of course, be obtained by graphic methods.

The case in question is that of the quay walls of the Empress Dock at Southampton.

The Empress Dock is a four-sided tidal basin, each side being about 800 ft. long. During construction the tide was excluded by means of an enclosing bank. It is made of concrete and is 30 ft. wide at base, 45 ft. high from cope to dock bottom, and 51½ ft. high from cope to foundation at back. It rests on a moderately firm sandy clay. The lower part of the wall is backed by the same clay in its virgin state, as it was built in a timbered trench. The upper part is backed with much the same material excavated from the dock and cast out and rammed in layers. The angle of repose of the clay was about 26 degs. After the backing had been raised to cope level and the ground in front of the wall had been removed, the north wall of the dock moved forward. The maximum movement was about 23 ft., but the wall maintained its upright position, showing that the movement was not due to any overturning effect, or to crushing of the clay underneath its toe, but simply to the fact that the resistance of the clay in front of the toe, plus the friction at the base, was insufficient to balance the lateral pressure of the backing.

This wall was taken down and rebuilt to the same section, except that the foundations were carried down to 15 ft. below dock bottom instead of 6½ ft., and the result has been quite successful.

At the same time, in order to try and prevent a similar accident to the other walls, large blocks or buttresses were built in front of the toe. These buttresses were each 20 ft. long, 15 ft. wide, and 12 ft. deep, the top of each being level with the dock

bottom, and the base 12 ft. below. They were spaced 30 ft. apart in the clear. It was thought that the buttresses being 6 ft. deeper than the wall could not plough up the ground in front of them, and would therefore resist its lateral movement. This hope was vain, however. Another of the walls (the east wall) moved forward, buttresses and all.

In order to save the west wall after this experience, besides building the buttresses in front of the toe, the backing was removed to a depth of about 13 ft. below quay level, and by this means serious movement was prevented until the water was admitted to the dock. The backing was not restored, the quay being carried on a viaduct. Even under these circumstances the quay was constantly showing slight signs of movement, and eventually elaborate strengthening works had to be undertaken, which need not be described here.

The conclusions arrived at show that calculations for the stability of quay walls cannot in our present state of knowledge be always relied upon, and that failures on the one hand, and waste of material on the other, are liable to occur even to the most careful and experienced designer. This, however, is not saying that such calculations should be ignored. On the contrary, they are most useful in suggesting means for increasing in the most economical manner the stability of designs which are known or suspected to be weak.

Devices for Increasing Stability of Quay Walls.—We will now consider some devices which have been used for increasing the stability of quay walls and inquire how far calculation and experience would lead us to depend upon them.

(1) *Admission of water into dock.*—Most quay walls are designed to have a certain depth of water in front of them, although in constructing such walls the water is often temporarily excluded, so as to cheapen construction. Calculations would indicate that the pressure due to this water is a great stabiliser, and experience undoubtedly bears this out. A notable instance is the case of the dock wall at Calcutta, described in papers by Messrs. Bruce and Apjohn in Vol. cxxi. of the *Minutes of Proceedings Inst. C.E.* The wall is 46 ft. high, and founded on a material which was said to resemble putty. In spite of the fact that the earth was left in front of the wall for a depth of 26 ft. above foundation, the wall began to move forward when the backing was raised to within 2½ ft. of cope level. The movement was arrested, however, by admitting water into the dock to a level of 42 ft. above foundation, and no further movement took place, even when 16 ft. more earth was dredged out from in front of the wall. Of course, if the earth had all been removed before water was admitted, as was done at Southampton, the wall would undoubtedly have collapsed.

It follows that during construction in the dry a certain amount of earth should always be left in front of a quay wall until the water is admitted, unless the wall is designed to be stable without any water in front of it.

(2) *Sinking foundations deeper.*—Calculations would indicate that the forces tending to resist sliding forward can be considerably increased by sinking the foundations deeper, and experience confirms this conclusion. For instance, as mentioned above, the Empress Dock wall which failed was successfully replaced by one of similar design, but carried down to 15 ft. below dock bottom instead of 6½ ft. The calculations on the deepened wall show that although the outward pressure on the earth backing was increased from 27 tons to 37·6 tons, the resistance of earth in front of toe was increased from 2·9 tons to 15·4 tons, and the friction at the base from 18·6 tons to 23·4 tons; consequently the balance of horizontal forces was as follows:—

Outward horizontal forces—

Pressure of backing 37·6 tons.

Inward horizontal forces—

Resistance of earth at toe 15·4 tons.

Resistance due to friction at base 23·4 "
38·8 "

Showing an excess of force tending to keep the wall

stable of 1·6 "

It is interesting to note that much the same balance on the safe side is obtained if Bell's formulæ are used, but the individual forces are greater.

(3) *Building buttress walls in front of main wall.*—Let us now consider the device which was adopted in order to try and prevent movement on the other walls of the

Empress Dock, after the north wall had failed. Buttresses of concrete 20 ft. by 15 ft. by 12 ft. deep were constructed in trenches in front of the wall toe, at intervals of 30 ft. It was thought, and calculations would seem to suggest, that the resistance to horizontal movement would be increased, as the buttresses were 6 ft. deeper than the wall. As a matter of fact they seemed to be useless in this respect. It is possible that in sinking the trenches for these buttresses the clay settled away slightly from under the wall, and that later water got into the void thus formed, reducing the coefficient of friction there. In any case, when the earth in front of the buttresses was forced forward by the movement of the wall, the buttresses themselves tilted backward and slid up the slope of rupture. It is evident that this could not have happened if the buttresses had been built in under the wall by underpinning, and probably if this had been done they would have been effective. In fact, the author has successfully stopped horizontal movement in a large retaining wall on clay by underpinning it in this fashion.

(4) *Making wall wider.*—A wall design which is suspected of being unstable may often be improved by increasing its width. This affects its stability in two ways. The extra weight of material induces increased friction at the base, and hence increased resistance to horizontal movement. And, again, the greater width of base gives a more even distribution of load on that base, and consequently reduces the reacting pressure tending to crush the earth under its toe.

It is of interest to notice that the base width of a high quay wall on an earth foundation is invariably much greater in proportion to its height than that of a small wall. Many small walls have been built with a base width equal to one-quarter of the height. But with high walls the widths are much greater.

It will be realised that in a gravity wall the chief use of the material of which it is composed is to impose weight on the base of the wall, and that the stresses on it are very low. Consequently the material itself should be of the cheapest kind. This consideration has led to the use of hollow monoliths or caissons of masonry concrete or reinforced concrete, which, when sunk, are filled with sand, rubble stone, or weak concrete. Some excellent walls of this type have been built at Glasgow, Avonmouth, and elsewhere.

(5) *Removing a portion of the backing.*—This is a device which has been frequently used to increase the stability of a wall which has been found to move after construction. Its effect is somewhat difficult to calculate unless the backing is removed right back to the slope of repose passing through the heel of the wall. There are cases where a wall has been saved from collapse by its adoption, notably at Southampton, where several of the older walls were treated in this way. It has the practical disadvantage that water generally finds its way into the void left by the removed backing, and tends to soften what is left, and thus to increase the pressure on the wall.

(6) *Improving backing.*—A far better device is to substitute a light and clean backing like ashes for the silts and clays which are so often the only materials readily available for this purpose. Ashes, even when charged with water, weigh only about 100 lb. per cu. ft., and have an angle of repose of about 35 deg. Had the Empress Dock wall been backed with this material, theory would indicate that the horizontal pressure per foot-run would have been 16.0 tons, instead of the 27.8 tons estimated for the sandy clay. One cannot, of course, verify such figures by experiment, but the author knows of a case of a quay wall, about 56 ft. high, and backed with clay, which started to move. The movement was arrested by removing the clay to a depth of about 10 ft. and substituting ashes.

(7) *Sloping base of wall.*—This device is one over which there has been much discussion. From theoretical considerations we would gather that provided the friction or resistance to horizontal shear were the same at the base of the wall as it is in the earth below, it makes no difference whether the base is sloped or level. In either case, if the wall moves it will shear the ground through a horizontal plane passing through the lowest point of the wall. But it seems likely that the friction or shear resistance is greater in the virgin earth than at the plane of junction between the base of the wall and the earth.

Experience of many walls on the sandy clay at Southampton bears out that a sloping base is more stable than a level one.

(8) *Driving piles under base.*—This is a device which was frequently adopted at one time, though lately it has rather gone out of fashion. The fact is, that bearing piles are useful for assisting a weak stratum to bear the weight of a heavy wall, but they are of

little help to resist horizontal movement. Probably in most cases, if well driven, the piles take the whole of the weight of the wall. Consequently the friction between the wall and the earth beneath it is nil, and the horizontal pressure from the backing is largely transmitted to the piles, which may fail by ploughing through the earth in which they are driven, and perhaps by breaking under the bending stress thus induced. For this reason, and also because they are difficult and expensive to drive inside a wall trench, their use has been largely abandoned. Both theory and experience suggest that, if used, they should not be driven plumb, but battered, so that the resultant of the horizontal pressure of the backing and of the vertical weight of the wall should be as nearly as possible parallel to the length of the piles.

Probably if piles are used under a gravity wall, the most effective way of placing them is to drive them as sheeting under the toe of the wall.

Although piles under a high concrete wall are probably of little use, some very excellent and economical designs are to be found in America and on the Continent, formed almost entirely of piles. In such a wall the horizontal pressure of the earth backing is transmitted by means of the front sheeting piles partly to the ground below dock bottom and partly to the decking which rests on the piles. The decking thus acts as an anchor tie, and is itself prevented from moving forward by the raking piles under its tail end. The latter piles should, of course, be driven at such an angle that they are more inclined to a vertical line than the resultant of the horizontal pull on the decking, and of the vertical weight of the earth above it. Besides thus acting as a tie to the sheeting piles, the deck serves to relieve them from the horizontal pressure due to the weight of the earth above it, as the latter is transmitted direct to the piles under the deck. To give this relief effectually, the deck should be wide enough to cover the slope of rupture of the backing, and to nearly reach the slope of repose. The sheeting piles must, of course, be strong enough to withstand the bending moment produced by the horizontal pressure of the backing. To reduce this pressure as much as possible, a specially chosen backing material, such as rubble stone or clean sand, is usually adopted.

It is probable that the vertical piles behind the sheeting, besides supporting the deck, serve to reduce the horizontal pressure of the earth backing, but to what extent they do so can only be guessed.

The author does not know of any case of failure of this class of wall, so that it is not possible to compare the results of calculation and experience.

(9) *Anchor ties*.—This has been, and is still, a favourite aid to the stability of quay walls. In the opinion of many engineers, however, they are of little use, partly because they are often placed so far apart that they can only bear a trifling proportion of the horizontal thrust, and partly because the weight of the backing which rests on them imposes on them a stress which alone is sometimes enough to break them. Probably the only case in which they are really effective is that of a jetty with two parallel walls back to back with earth between them. In such a case the two walls may be economically tied to each other with steel rods.

(10) *Lengthening toe*.—In cases where it is suspected that a wall will crush the material under its toe, the obvious remedy is to lengthen that toe. The White Star dock wall has a toe 9 ft. long, which was added in order to reduce the pressure under this heavy wall to about 4 tons per square foot. In this case the toe was reinforced with old rails in order to prevent its being broken off from the body of the wall.

As previously mentioned, it is quite difficult to find instances of quay walls which have failed by overturning, so that the value of this device can only be conjectured.

Conclusion.—In conclusion it may be said that owing to the difficulty and expense of making experiments on large walls, it is the more important that careful records should be kept and published of both successes and failures in this class of structure. Such records should include not only dimensions of the wall, and careful notes as to the nature of the materials employed, but also the calculations for stability, with the formulæ used, the assumptions made for the values of constants in those formulæ, and the reasons for such assumptions.

It is, of course, unlikely that it will ever be possible to design large quay walls purely from rules and without some measure of judgment, but if a body of information, as suggested above, could be collected and analysed, it would help to clear away some of the uncertainties which now beset that interesting though troublesome problem—viz., the most economical design of high quay walls on earth foundations.

DISCUSSION.

The President, dealing with the difficulties that encompass an engineer in designing quay walls for a modern deep dock, said that, many years ago, he saw a dock wall being constructed and predicted its failure by overturning, which, however, did not occur until some twenty years later. The original sloping bank behind the wall was simply filled in without being punched, the consequence being that undue pressure was brought on the wall. A Great George Street engineer was called in, and a new wall built up on cylindrical caissons sufficient to defy an earthquake, and at proportionate cost. Failure by sliding forward occurred in another case with a properly constructed wall through dredging too near the toe of the wall. The case of the sliding wall at the Empress Dock described by the author was particularly interesting, and the only remark he had to make upon it was that, apart from going deeper, he should have made the base slope backwards so that the wall would have to lift before it could slide forward. Another explanation of the failure which occurred to him was that the pressure of two to three tons at the toe upon the moist earth at the bottom of the dock caused the material to spew up in front and so reduce the resistance to sliding. He always used Rankine's formula for retaining walls, even for clay, but the main question was, what was the natural slope of clay? In his experience, when the clay became moist it acted more like a viscid fluid and it was hardly possible to say that it had any real natural slope. When it was dry it was virtually a different material altogether. He had always considered the line of rupture as more like a semi-parabole with a vertex on the surface, and that would be seen in any section immediately after a slip had occurred. In the case of water getting behind the wall, Sir Benjamin Baker had showed that the pressure was greater than if the backing were water only. At the Victoria Docks, London, anchor ties used between a pair of quay walls exhibited signs of incipient failure, and that was found to be caused by the ties rusting through. Fortunately, this defect was discovered in time to prevent a disaster.

Mr. Charles F. Marsh, M.Inst.C.E., did not think there was any real advantage in sloping back the foundation of a wall, and particularly in clay, as clay would shear through owing to the horizontal pressure behind the wall. A very interesting paper by Mr. Neames in the *American Society's Journal* showed that the pressure was entirely due to the parabolic curve of earth failure at the back of the wall. In his own experience he knew of two very well defined cases of failure on reservoir walls in London clay. In both cases the walls slid forward: there was no sign of any overturning. When a big wall was designed with a clay backing and on a clay foundation, undoubtedly the main thing to look after was the sliding forward on the base. If much wet weather occurred during the construction of the wall the water got behind or round the wall and it sucked in underneath, and it was frequently found that there was practically no coefficient friction at all between the wall and the foundation. Recently a reservoir had been designed in the London clay, with a tongue underneath 3 ft. wide with 10 ft. of reinforcement on rolled joists, vertical and horizontal bars. That wall had not yet been constructed, but he believed it would be perfectly safe and would not slide forward. Personally, he should be perfectly satisfied with a factor of safety of 1.1. He was not an expert in dock construction, but he thought that dock walls ought to be designed without taking any account of the water in front, because it was obvious that the wall must be constructed before they let the water in.

Mr. Morgan E. Yeatman, M.A., regarded the lateral pressure of earth-work as the most vexed question in all engineering design. A good deal depended on whether the wall and the foundation under the wall were pervious or impervious. The class of quay walls with which he personally had had most to do were walls built of concrete blocks laid under water on a broken rock rubble foundation. In that case both the wall and the foundation were to a great extent pervious to water, and it was necessary to assume that they had the same level of water behind the wall as in front of it; but it followed from that that they must take the weight of the lower part of the wall below the water level as measured in water and not as measured in air. With regard to the failure of the wall referred to, the curious thing was that the wall acted as if the pressure were behind the centre, and pushed down behind, but, according to all theories of calculation, before failure began the pressure on the base was in front of the centre. That was why it was hard to understand. He differed from Mr. Marsh, because undoubtedly the friction of a flat, smooth bottom wall on the clay was less than the shear strength of the clay on itself. The valuable point in the author's paper was that the danger in most cases seemed to be more from sliding than from overturning, and the safest way to avoid that was to go down deeper into the foundation.

Dr. J. S. Owens, B.A., M.D., said it seemed to him that most methods of measurement did not distinguish between what he might call the adhesion and the friction, that was the force

required to start a body moving and the force required to keep it moving when it had started. The crux of the problem was the determination of the constants, the angle of repose of the material. It was really impossible to do that, and the sooner they recognised it the better. In connection with clay, fine or wet weather and the effect of vibration were variable factors which one was liable to forget. It was worth bearing in mind in making a big excavation that there might be some larger movements of the earth or clay than the movements in the immediate vicinity of the wall, and he suggested that this was a subject pre-eminently for experiment, which might be undertaken by the Institute.

Mr. S. Bylander (*Past Chairman J.I.B.*) gave particulars of an ordinary retaining wall 60 ft. high for which he was responsible, and said that from his experience in designing such walls, he had come to the conclusion that, with a relatively low wall, they must add a surface load.

Mr. W. G. Perkins (*District Surveyor for Holborn*) remarked that the effect of springs of water in the earth would tend to lift the wall in the way suggested by the President, and the water would make the clay slippery because it would act as a lubricant.

Mr. Percy J. Waldram, F.S.I., was particularly familiar with the failure of walls in graving docks due to the lift of the earth in front of the wall, which had not been taken into account in the paper. Many such walls were built with an inverted arch for that particular reason. He dissented from the statement, frequently put forward by people who ought to know better, that widening the base gave more friction. It would certainly seem to be desirable that anchor ties should always be imbedded in a small piece of concrete. They must have a trench to put them in, and it was far better to fill it up with concrete than to leave the ties exposed to the action of the soil.

Mr. J. Ernest Franck, A.R.I.B.A., recalled his visit to Southampton, when he was struck with the enormous size of one of the walls which the Author was constructing there, on which occasion he remarked to Mr. Wentworth-Sheilds that when London had disappeared in the next 100,000 years, whoever came to make excavations on this site would at any rate find something worth digging out.

MR. WENTWORTH-SHEILDS'S REPLY.

In replying to the observations of the President and Mr. Yeatman, the Author said there were many cases in actual practice where the shear resistance of the material itself below the wall was greater than the friction between the wall and the material on which it rested, that was to say that the actual plane of the base of the wall where the concrete touched the clay was the plane of least resistance, and therefore he contended that it was right to make that an uphill plane. There might be something in what Mr. Yeatman had said, that the movement of the Empress Dock wall was due to the fact that the earth tended, as it were, to depress behind the wall and to rise up in front of it, but he inclined to the opinion that what largely accounted for that curious movement was simply that a wedge-shaped piece of earth moved in front of the wall, forming a plane of rupture. The wall itself also tended to move up that plane, and in doing so it naturally overturned backwards. One wished it were possible to ascertain by experiment the actual pressure of different kinds of backing, such as dry gravel, gravel charged with water, and so on. If that could be done with any chance of success it would be worth the while of the Institute to undertake such experiments. In dealing with clay they could only assume some such value for their constants as he had tried to suggest in the paper in order to get something that was reasonably likely to be safe and not too outrageously extravagant.

Mr. Perkins inquired whether the direction of the pressure was horizontal or parallel to the angle of repose.

Mr. Wentworth-Sheilds admitted that that was a very difficult question to answer, but when considering the stability of a wall against sliding forward he was of opinion that the lateral pressure was wholly horizontal. On the other hand, he believed it was right, in considering the stability of a wall as regarded overturning, to assume that there was a vertical component as well as a horizontal component to the lateral pressure: in other words, that the pressure was inclined. Although text-books suggested that they should go in for factors of safety of three and four, he considered it absolutely hopeless to try with economical design to get such a factor of safety. It could only be done by what might be called unwarranted extravagance in wall design, because the quantities of material which would have to be used to get such high factors of safety would be quite unnecessary, and indeed in some cases it was doubtful whether such a factor could be obtained at all.

NEW WORKS IN CONCRETE AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

CLECKHEATON GAS WORKS.

THE Cleckheaton District Council have recently erected a new gas works at Moorend, and various portions of the work, including the foundations to the retort house, an underground tar and liquor tank, a small water tank, and three substructures supporting a truck weighbridge and two coal hoppers were constructed in reinforced concrete. It was decided to have a raft foundation to the retort house in order to reduce the



Timber Sheetting of top of Reinforced Concrete Tar and Liquor Tank, 40 ft. dia.
(to prepare for concreting).
CLECKHEATON GAS WORKS.

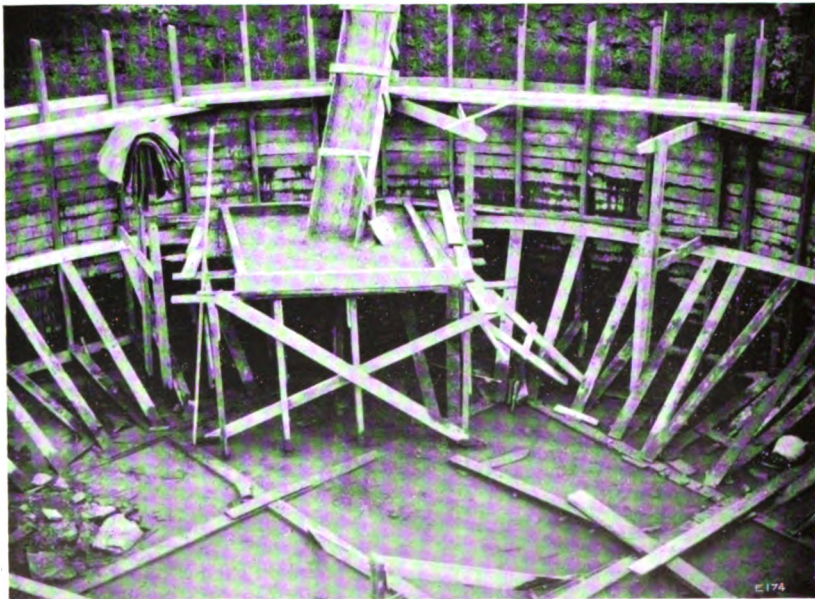
pressure on the ground to a maximum of one ton per sq. ft. The siding was constructed by tipping 45,000 tons of material to a depth of 32 ft. along the length of the site adjoining the L. & Y. R. main line, and the substructures were carried from the solid ground to prevent sinking of the weighbridge and coal hopper by subsidence of the tipped material.

The retort house raft is about 85 ft. long by 40 ft. wide, with four longitudinal rows of columns at distances of 10 ft., 16 ft., and 10 ft. centres, respectively. The columns in the two external rows are at 15 ft. centres, the load on each column being 5 tons, and the columns in the two internal rows are at 7 ft. 6 in. centres, the load on each column being 90 tons.

The principle on which the raft was designed was reinforced concrete slabs spanning between longitudinal reinforced concrete beams under each row of columns. The beams and slabs were reinforced with Kahn trussed bars and rib bars.

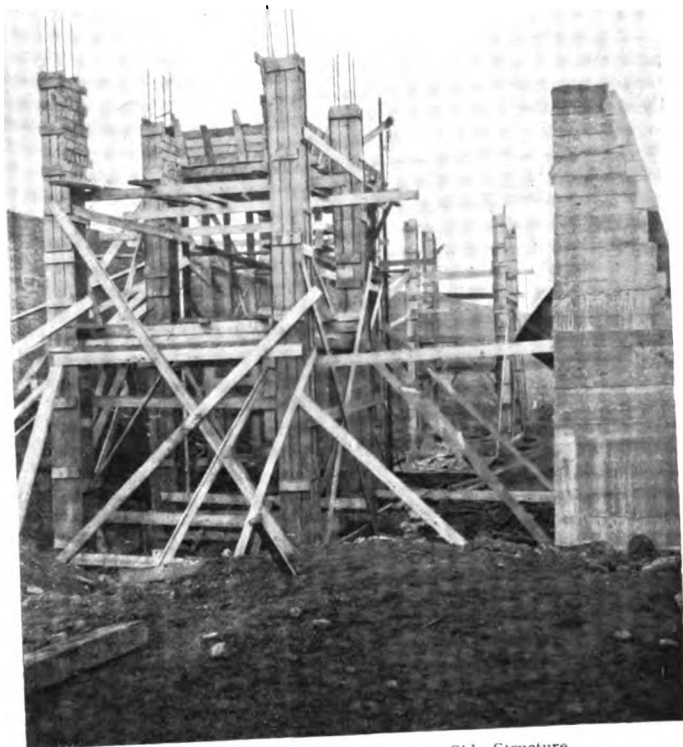


Reinforced Concrete Raft for Retort House.



View showing the forming sides of Reinforced Concrete
Underground Tar and Liquor Tank.

CLECKHEATON GAS WORKS.



Shuttering for Reinforced Concrete Side Structure.
CLECKHEATON GAS WORKS.

The liquid residuals of the process of gas manufacture are collected in an underground reservoir, which is also constructed in reinforced concrete. The tank is a circular one, with an internal diameter of 40 ft., and holds a depth of 12 ft. 6 in. of liquid. Its capacity is 99,000 gallons. The bottom slab and sides of the tank are 7 in. thick, and the roof, which carries 4 ft. of earth, is composed of a 5-in. slab spanning between beams which are carried on the outer wall of the tank and on two internal columns. The whole of the reinforcement consisted of Kahn trussed bars and rib bars.

A small underground water tank, 13 ft. 6 in. long by 11 ft. wide and 7 ft. 5 in. deep, and divided into two compartments, was also constructed in reinforced concrete. The bottom slab and walls of the tank were 6 in. thick, and the roof 5 in. thick. Kahn rib bars were used as reinforcement throughout.

Three substructures to an elevated weighbridge and two coal hoppers are also constructed in reinforced concrete. They consist of a reinforced concrete platform slab which is carried on four external beams supported on four corner columns about 24 ft. long, with horizontal braces midway between the bases and the platform for the former, and similar columns for the latter. The column bases are in reinforced concrete, and are 4 ft. square.

Other plant and buildings erected on the new works are boiler, pump, exhaustor, washing plant, meter and governor houses, in separate houses, purifiers, condensers, and overhead tanks. The works have been laid out on an ornamental scale, and have cost £42,000.

The whole of the works, with the exception of the retort house, were designed by Mr. A. L. Jennings, gas engineer and manager, with Messrs. Thomas Newbigging and Son, of Manchester, as expert advisers.

The whole of the reinforced concrete work here described was designed on the Kahn system of reinforced concrete by the Trussed Concrete Steel Co., Ltd., of Caxton House, Westminster, S.W.

We are able to publish these particulars by the courtesy of Mr. Jennings, who also kindly placed the photographic illustrations at our disposal.

**REINFORCED
BRIDGES,
COVENTRY
LOOP LINE,
LONDON AND
NORTH
WESTERN
RAILWAY.**

THE new loop line which has just been laid on the L. & N.W. Railway system near Coventry, for goods traffic, is only about three and a half miles long, but there are no less than fourteen bridges, the construction of which was necessary in the formation of the track. Of this number, eight are executed in steel, and one in brickwork, while the remaining five

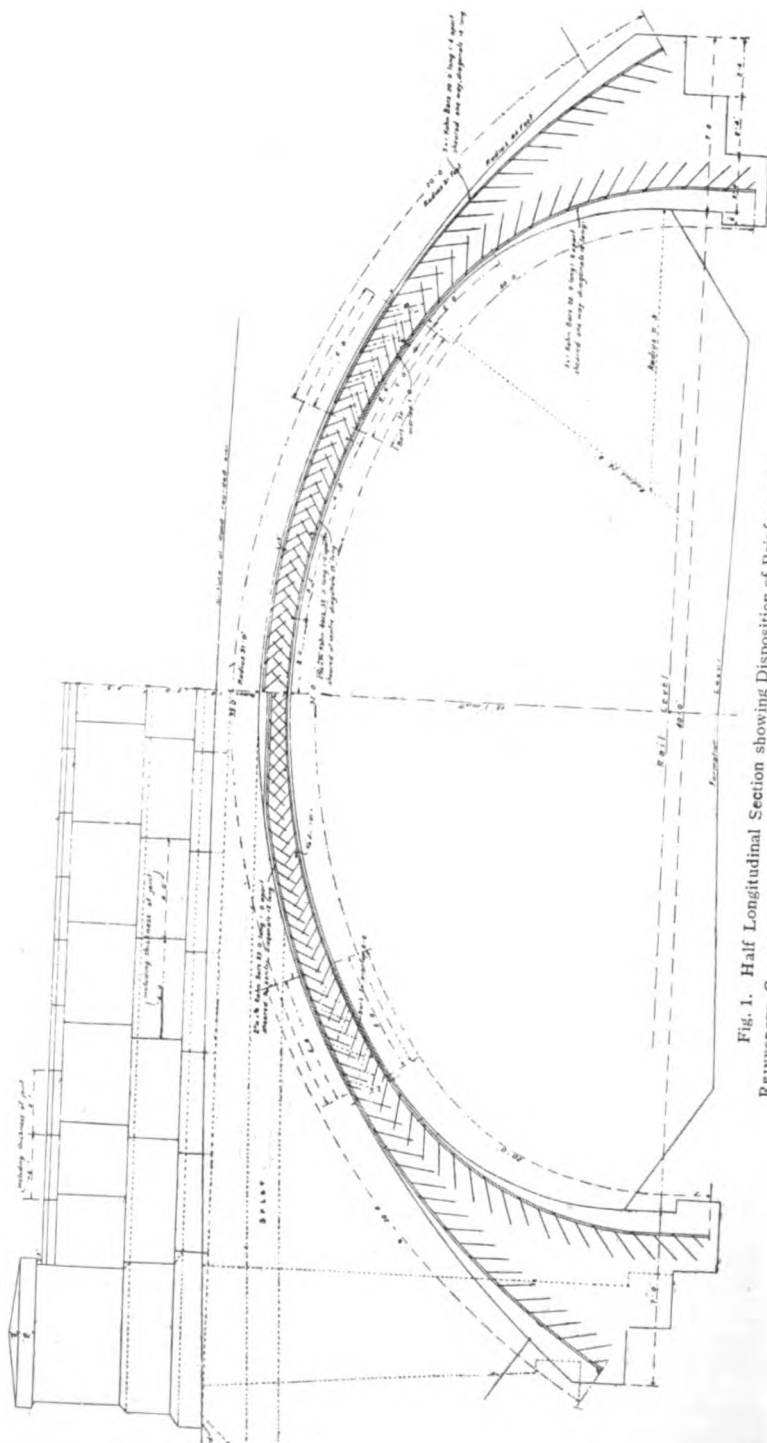


Fig. 1. Half Longitudinal Section showing Disposition of Reinforcement.
REINFORCED CONCRETE BRIDGES ON THE COVENTRY LOOP LINE, L. & N.W. RAILWAY.

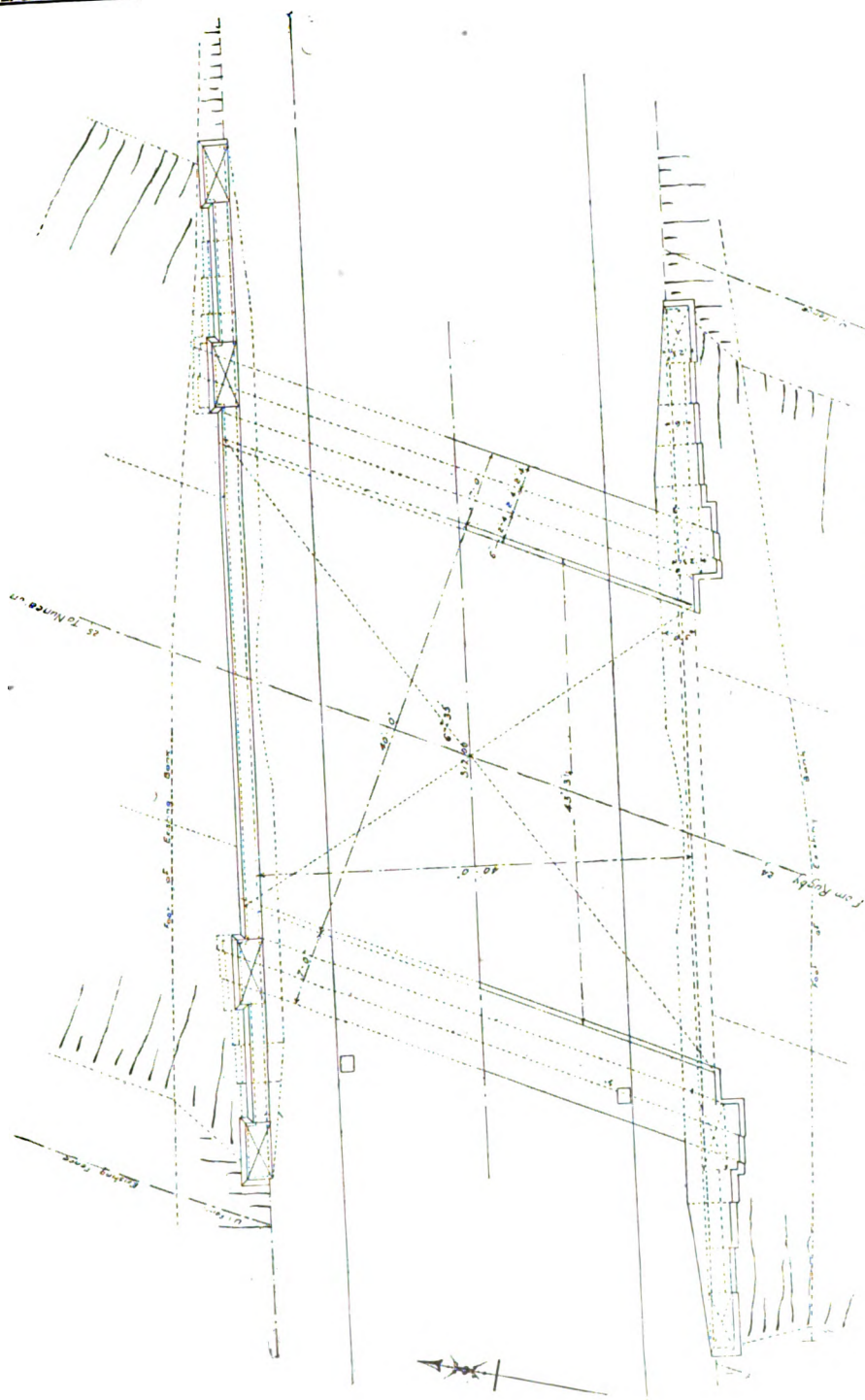


Fig. 2. Half Plan of Superstructure and Foundations.
REINFORCED CONCRETE BRIDGES ON THE COVENTRY LOOP LINE, L. & N.W. RAILWAY.

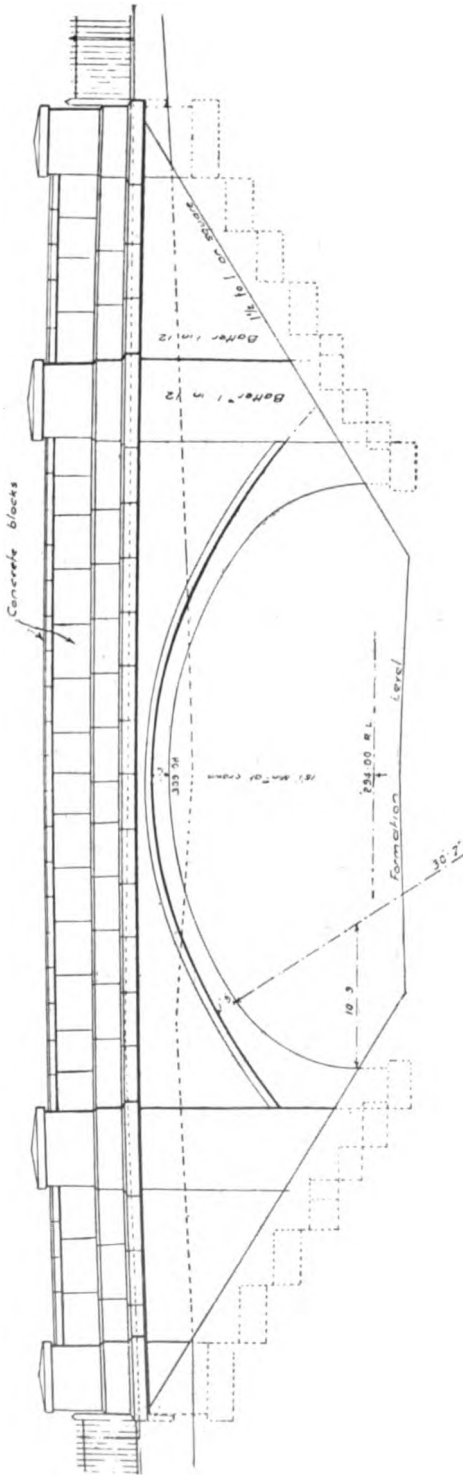


FIG. 3. Elevation.
REINFORCED CONCRETE BRIDGES ON THE COVENTRY LOOP LINE, L. & N.W. RAILWAY.

carry streets or roadways over the railway track, and these are built of reinforced concrete, some details of which are here given. In all cases a width of 40 ft. between the parapets was allowed, and a similar style was adopted both in construction and design, with the necessary modifications to suit the conditions.

In three instances the roadway carried by the bridge is at right angles to the railway track, and in consequence these are simpler than the remaining two, where the arches had to be of the skew type. The illustrations in Figs. 2 and 3 show the plan and elevation for one of the skew arches. The arch has a clear width of 40 ft. at the springing, the latter being situated at a level of about 3 ft. 6 in. above the formation level of the track, and the soffit is set out as a three-centred curve, with a radius of 10 ft. 9 in. at the abutments and 30 ft. 7 in. at the crown, a minimum of 15 ft. 1 in. being given above the rail level, at the crown. The arch ring has a thickness of 1 ft. 3 in. at the crown, and this increases towards the abutments, as the extrados does not follow the same curve as the soffit, but is struck from one centre only, and this has a greater radius than the latter at the crown. A section showing the disposition of the reinforcement is given in Fig. 1, and it will be seen that Kahn bars were used in both soffit and extrados, these being spaced about 12 in. apart in the straight bridges and about 10 in. in the skew bridges, with diagonals 12 in. or 18 in. long according to the position.

The bars were placed in three lengths, those at the extrados being 20 ft. long in the lower part of the arch and 33 ft. long for the central portion, these bars having a lap of 5 ft. The bars in the soffit are similar, with the exception of those in the central portion, which are only 32 ft. long. A minimum cover of 2 in. of concrete is given to these main bars. Longitudinal rods, $\frac{5}{8}$ in. diameter, are placed in the soffit at intervals of 3 ft. to 5 ft., and these are hooked over the main bars at the ends. The arch abutment is carried well down into the side of the cutting, and has a width of 7 ft. with off-sets as shown. The piers, on either side of the arched opening, and the wing walls have a batter of 1 in 12, and these are formed in concrete, while the parapet, which is about 6 ft. high, is built of concrete blocks. The whole of the work was designed and executed from drawings prepared by Mr. E. F. C.

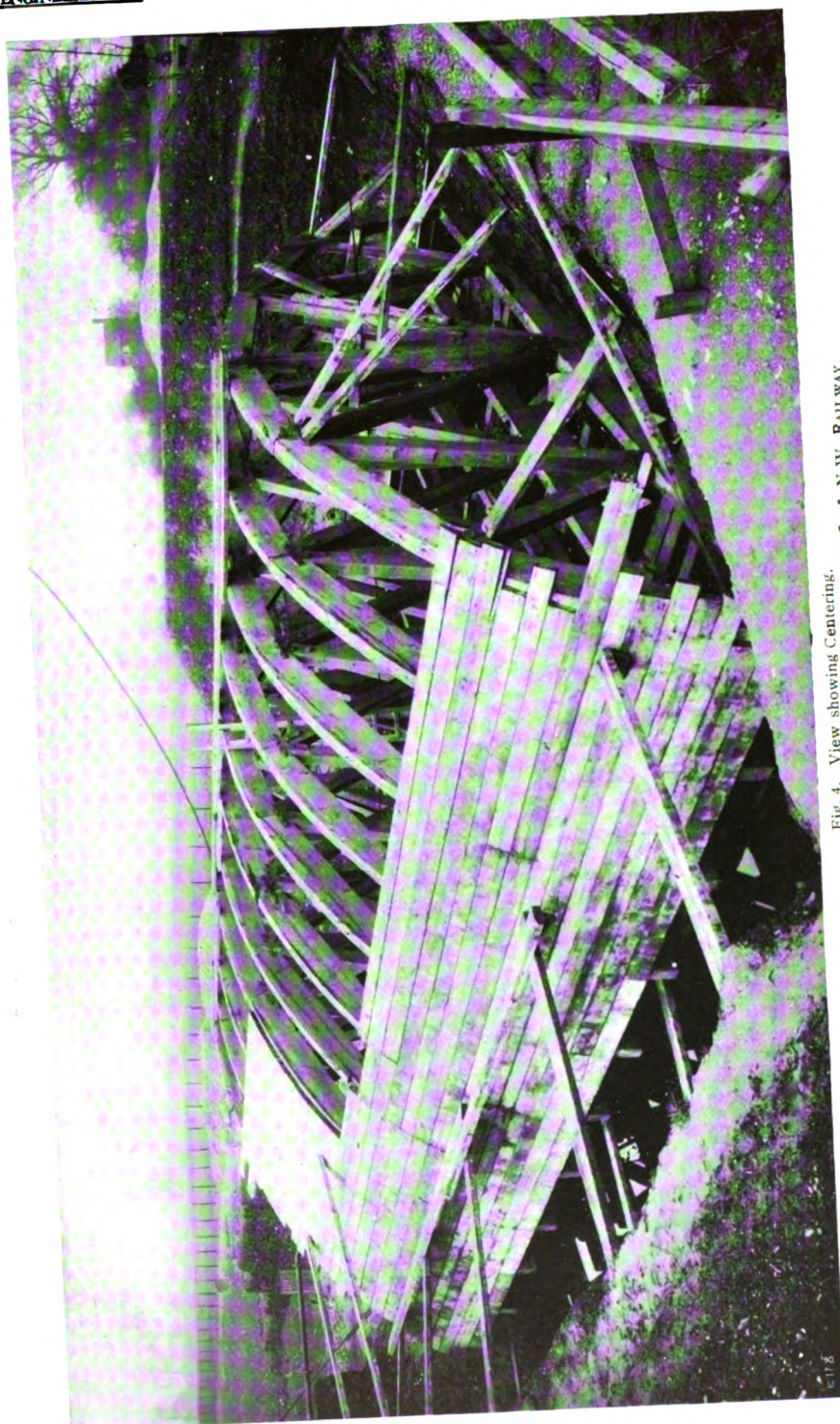


Fig. 4. View showing Centering.
REINFORCED CONCRETE BRIDGES ON THE COVENTRY LOOP LINE, L. & N.W. RAILWAY.



Fig. 5. View of One of the Finished Bridges, L. & N. W. RAILWAY.
REINFORCED CONCRETE BRIDGES ON THE COVENTRY LOOP LINE, L.

Trench, the Chief Engineer to the London and North-Western Railway; and it may be considered as a good example of the economical application of reinforced concrete, and as practically no maintenance will be required on these bridges the adoption of the material is very wise.

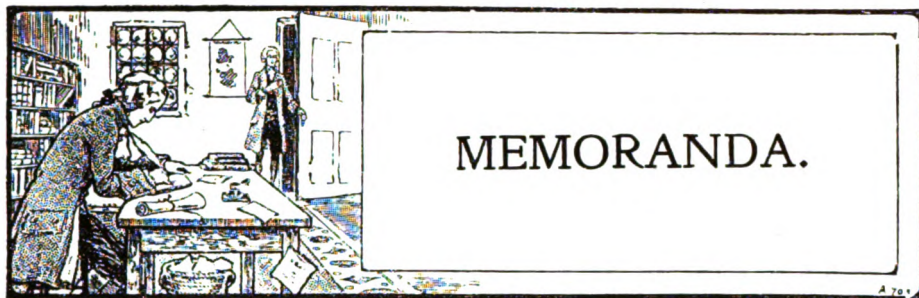
We are indebted to the Chief Engineer of the L. & N. W. Railway, Mr. E. F. C. Trench, M.Inst.C.E., for our illustrations.

REINFORCED CONCRETE CHURCH, HASTINGS, N.Z.

Our frontispiece shows a church erected in reinforced concrete in New Zealand to the designs of Mr. F. de J. Clere, F.R.I.B.A.

Reinforced concrete is particularly suitable for buildings in New Zealand, as the country is subject to earthquakes, and although no serious shocks have been experienced for many years, such as have occurred have been known to cause serious fractures in brick walls and many chimneys to fall.

We hope later on to be able to publish some detailed particulars of the building here illustrated.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Reinforced Concrete Sleepers for Tramways.—The *Commonwealth Engineer* contains an article on reinforced concrete sleepers, from which we give the following extract :—

“ The Adelaide municipal tramways are regarded as one of the most modern electric systems, and under their progressive method it is not unusual to find that as new ideas are introduced this system is one of the foremost in testing them. The tramways recently inquired into the stability of reinforced concrete sleepers, and have just completed a running test over a period of six months on two of their suburban tracks.

“ These tests have proved that the reinforced concrete sleeper has come to stay, and it is claimed that the tracks in which they are used are the easiest running. Experiments have been carried out to record the amount of oscillation made by carriages running over tracks in which the reinforced concrete sleeper and the ordinary wooden sleeper are laid, and the record thus made shows a vast comparison in favour of the reinforced article.

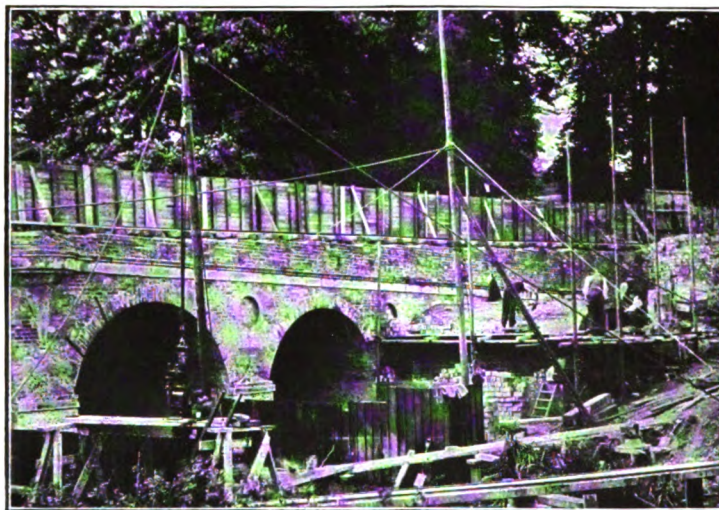
“ The sleepers used are the Joseph Timms reinforced, consisting of a form of reinforcement, in conjunction with a metal shoe on which the rail sits. The shoes are each connected with reinforcing rods, which are strengthened by intervening braces, the whole combining in a truss bridge, which in itself is sufficiently strong, but is further strengthened by the addition of the concrete. In the manufacturing the ironwork of the sleeper is placed in a mould, into which the concrete is poured, and the necessary tamping is done to ensure that every available space is filled. The bottom of the mould is made somewhat convex, so that the bottom of the sleeper when it issues from the mould has a concave surface. This is regarded as being an improvement on the usual flat bottom, as it has a greater tendency to grip the bed in which it is laid.

“ When the sleepers are laid the rails do not come into contact with the concrete, and all vibration is taken on the shoe. Thus any possibility of the concrete fretting is obviated. The metal shoe grips one side of the foot of the rail in a solid clamp, while the opposite rail is clawed by a locking-bar and forced against the foot of the rail by two wedges inserted from opposite sides. By this means the gauge may be altered up to the thickness of the wedges, but when these are firmly driven in, the rail and sleeper become one compact mass, without the possibility of the least movement. At the end of the test the track was opened in several places, but in no instance could any movement be observed, and in no case was it found necessary to tighten up any of the wedges used in attaching the rails to the sleepers.

“ Particularly in towns should the reinforced concrete sleeper be utilised, as once they are laid they are there for years, thus saving the inconvenience which always occurs where tracks have to be opened up in thickly populated centres. And in districts which are infected with white ants their value is apparent.

“ While the initial cost per sleeper is greater than wood or steel, its advantages are so many that its general use would really result in a big saving. It is difficult to prophesy exactly how long its life is, because concrete usually improves with age, but it is apparent that it is very much longer than that of wood, and once laid is far and away cheaper to maintain. Several of the older countries of Europe have realised this, and at the present time the authorities controlling the Rome-Naples railway are replacing the steel sleepers with the reinforced concrete.

THE BRITISH STEEL PILING CO.



WIDENING COBHAM BRIDGE.

Right of picture shows a "Simplex" Piling (22 lbs. section) cofferdam being withdrawn (after "founds" of some arches were completed) by means of a pole, a set of blocks, and one of our "Grips." Left of picture shows same piling being redriven with a No. 3 McKiernan-Terry Hammer suspended from a pole to form another cofferdam—no pile-driving frames or complicated plant required. Only sufficient piling for $\frac{1}{3}$ rd of total foundation area ordered; used three times, then sent on another job.

COULD ANYTHING BE SIMPLER?

Weights of Piling on sale or hire :—

"SIMPLEX"	- -	22 to 27 lbs. per sq. ft.
"UNIVERSAL JOIST"		from 43 lbs. "

DOCK HOUSE, Billiter Street, LONDON, E.C.

"It may be argued that reinforced concrete sleepers become useless if a change of gauge is contemplated. This may in certain designs be the case. But in the sleeper used at Adelaide provision is made which enables a change from a broader to a narrower gauge, or vice versa, to be undertaken most easily, and at a very small cost.

"The highly satisfactory results obtained in South Australia have induced the Government of the State of New South Wales to undertake exhaustive trial tests on the railways.

New Devon Bridge.—The official opening of the new Umberleigh Bridge has just taken place. The bridge, which is the only one of its kind in the West of England, is of strongly built reinforced concrete arches, the walls and abutments being of mass concrete. There are three arches of 50 ft. span of reinforced concrete. The work, which cost about £2,700, was carried out by Messrs. Pollard and Co., contractors, of Taunton. It was supervised by Mr. R. M. Stone, the county surveyor.

Reinforced Concrete Picture Houses.—Plans have been submitted to the Bristol Sanitary Committee for a reinforced concrete picture house at Redland. This form of construction was not contemplated when the bye-laws were framed, but the Committee have decided as a general rule that reinforced concrete shall be allowed subject to the details being satisfactory.

Loans for Public Work.—In view of the uncertainty which has prevailed amongst local authorities as to the policy of the Treasury in allowing loans for public works, the statement made by Mr. Montagu, the Secretary to the Treasury, in the Parliamentary Papers will be welcomed. Mr. Montagu states that at the present time loans are only granted by the Public Works Loan Commissioners where it can be shown that their purpose is urgently necessary, in the national interests, for the furtherance of the War, or is of urgent necessity for reasons of public health. This arrangement has been made in pursuance of the Government's general policy of postponing or avoiding all new capital expenditure wherever possible, in view of the paramount necessity of conserving the capital resources of the country in the national interests. New loans are therefore only granted in respect of works for which a local authority is able to produce evidence of Treasury or Local Government Board approval, granted since the special restrictions arising out of the needs of the War have been effective. In cases where loans have already been granted, but either have not yet been advanced or have been only partially advanced, the Public Works Loan Commissioners require that all possible efforts must be made by the local authority concerned to postpone or delay expenditure in connection with the loan, and advances are not made out of the loan unless it can be shown that these efforts have been made.

PROPOSED NEW WORKS.

Canada.—Harbour works are being carried out at Port Arthur and Fort William. A breakwater, 20,800 ft. long, is to be constructed at Port Arthur. At Fort William the revetment wall is to be extended at a cost of 365,000 dollars. The Department of Public Works contemplates in the future the extension of Bare Point breakwater.

Brierfield.—The Urban District Council propose constructing retaining walls at the sewage disposal works at an estimated cost of £2,500.

Dunbar.—It has been decided by the Town Council to carry out important repairs to the north wall of the new harbour in order to protect the structure from further havoc by the force of the sea.

Motherwell.—In connection with the proposed erection of a coal store for the Gas Department, the Corporation have invited an alternative scheme for a brick building, with reinforced concrete roof.

Aberdeen.—The Harbour Board have decided to curtail the extension of the Provost Meerns Quay, owing to lack of materials and scarcity of labour.

Colchester.—Amongst the works for which tenders have been invited during the month has been that for the construction of a reinforced concrete tank to hold 100,000 gallons of water at Severalls Asylum, Colchester, for the Committee of Visitors.

Reading.—The Town Council have been informed by the Treasury that the borrowing of the amount required for the construction of the reinforced concrete bridge in De Bohun Road must be deferred for the present.

Tregaron.—The Rural District Council have decided to erect a small reinforced concrete bridge over the river Ystwyth.

Brighton.—The Corporation have agreed to make further contributions towards the sea defence works between Black Rock and Newhaven.

Queensland.—A high-level reinforced concrete bridge will be built over Moores Creek by the North Rockhampton Council, Queensland.

Dumfries.—In connection with a scheme for the improvement of the navigation of the river Nith, Mr. W. A. Tait, Engineer of the Board of Agriculture of Scotland, recommends the erection of a leaning wall on the Dumfriesshire side.

Rhondda.—Tenders have been invited by the Urban District Council for the erection of a cast-iron or reinforced concrete water tank with a capacity of 30,000 gallons.

Dundee.—The Town Council have recently invited tenders for reinforced concrete floors, etc., at the extension of their electricity generating station.

Seville.—The *Gaceta de Madrid* publishes a Royal decree authorising the Junta de Obras of the Port of Seville to negotiate a loan of £160,000 for the carrying out of improvement works at the mouth of the river Guadalquivir.

TENDERS ACCEPTED.

In the past month the War Office have accepted the tender of Messrs. W. Bain and Co., Lochrin Iron Works, Coatbridge, for reinforcing columns; and of the Croft Granite, Brick and Concrete Co., Ltd., of Croft, near Leicester, for the supplying and laying of concrete flags at Larkhill.

The Kirkcaldy Town Council have accepted the tender of Mr. R. Henderson, Edinburgh, for constructing a concrete toe at the sloping wall in the tidal basin of the harbour.

The Corporation of Birkenhead have provisionally accepted the tender of Mr. J. Riley, of Cheltenham, of £61,323, for the construction of a portion of the aqueduct in connection with the proposed Alwen water supply.

ERRATUM.

In our April issue we regret that an error was made in the article on the New Pier No. 2, Halifax; page 201, line 20, the depth of the upper girders should read 50 in., and not 30 in., and on page 203, line 12, C.D. should read C.B.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

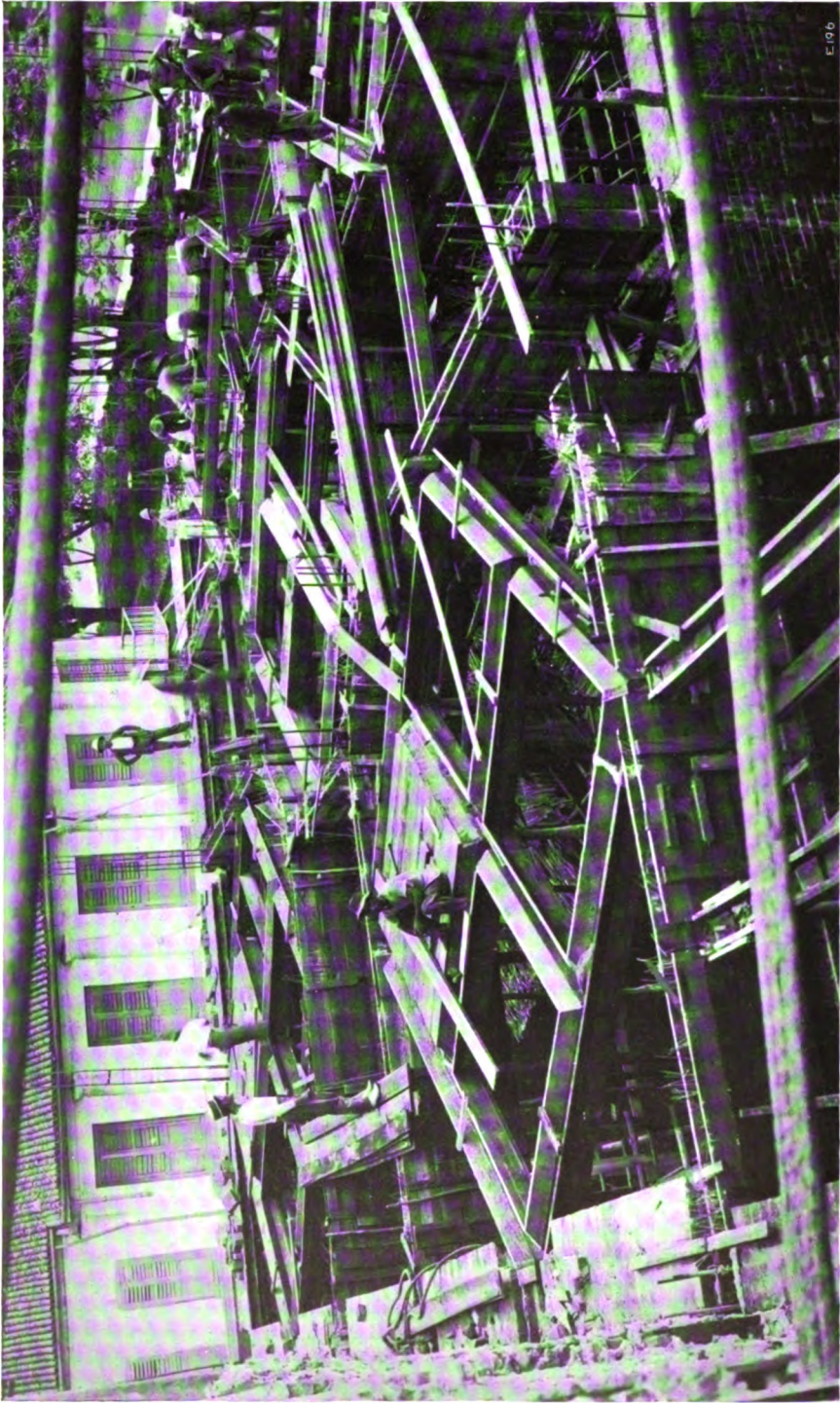
The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



THE NEW OFFICES FOR THE INDO-CHINA BANK.
(In course of construction.)
(For *Description*, see p. 420.)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X. No. 8.

LONDON, AUGUST, 1915.

EDITORIAL NOTES.

L.C.C. REGULATIONS FOR REINFORCED CONCRETE CONSTRUCTION.

THE revised regulations for Reinforced Concrete Work were submitted and approved at a recent meeting of the London County Council, and we would refer our readers to the Memorandum on page 426, which deals with this subject. The revisions that have been made since the second set of regulations was prepared are so extensive that they assume great importance to all designers, and we therefore consider it necessary to call attention to the principal alterations in these notes. Owing to lack of space, we are unable to publish the actual regulations as now approved by the Council in this issue, but hope to print them in the next two numbers, and in the meantime these notes of the alterations that have been made will indicate their nature and effect. Generally speaking, the regulations have been amended in such a manner as to allow greater latitude in the design and construction, and this is no doubt due to the efforts of the Institutes which are mentioned in the General Powers Act, and who have taken up the subject in earnest with the idea of obtaining conditions that are fair. The Concrete Institute has certainly done good work in this respect, and although some members of the Institute are averse from regulations altogether, we feel that the majority look at the subject in a more reasonable manner, and realise that laws are unavoidable, and, indeed, are desirable if framed in a proper manner.

GENERAL.

Part I. of the regulations, which deals with the general clauses and scope of the subject, has been revised in one or two details, but the scope is still confined to buildings in which the loads and stresses are transmitted through each storey by a skeleton framework of reinforced concrete, with the assistance of party-walls only, and it has not been extended to apply to reinforced concrete which is used for parts of a structure only, although there is no doubt that the rules made will be applied in the latter case by District Surveyors, who necessarily require some standard as a basis. The clause relating to floors and stairs has been extended, and now includes roofs; but the inclusion is in the form of a statement which permits the use of wood-framing, boarding, and battens in the construction of roofs, notwithstanding the fact that all floors, stairs, and landings, together with their supports, must be executed with incombustible materials. A new clause is added to the effect that no part of the reinforcing metal shall be used for conducting electrical currents.

DATA.

The data to be used in the calculations is specified in Part II., and the dead loads per foot super to be provided for in the different classes of buildings have not been varied; but a new clause has been added, which states that stairs and landings shall be calculated to carry safely a dead load of at least 120 pounds per square foot, but each step shall be capable of safely supporting a concentrated load at any point of not less than 300 pounds, and this is rather a necessary addition, as no definite data was given in the previous rules. Another new clause is introduced, which stipulates that "for calculating the resistance moment the angle of dispersion of a point load through hard filling and concrete shall not be taken at more than forty-five degrees from the vertical." This, we think, would be generally accepted and adopted by designers; but it is as well to get a definite regulation which must be worked to. The notes on wind pressure remain unaltered, although the allowances made are, no doubt, excessive. A slight variation is made in the wording of the definition of "working load," and the finishing materials, such as plastering, etc., are now stated to be included, and this has affected the following clause, wherein it is specified that for the purposes of calculation the average weight of reinforced concrete, together with the finishing materials, shall be taken at not less than 144 pounds per cubic foot measured over finished surfaces. In the previous regulations the weight to be taken was 150 pounds per cubic foot; but this related to the reinforced concrete only, and the finishing materials had to be considered as a separate item. This alteration is advantageous, as it simplifies the process of finding the total weight to be carried, and it certainly does not err on the side of increasing this total load.

An important revision has been made in the regulation relating to the ratio of span to depth of a beam. In the previous rules it merely stated that the span should not exceed twenty times the effective depth; but this has now been put on a more satisfactory basis by making the ratio dependent, to a certain extent, on the comparison of the actual stress in the material to the permissible stress, and it is stated thus:—"The ratio of the span of a beam to its effective depth shall not exceed the lesser of the two following ratios—

$$20 \times \frac{\text{tensile stress in regulation 43}}{\text{actual maximum tensile stress}}$$

or

$$20 \times \frac{\text{compressive stress in regulation 42}}{\text{actual maximum compressive stress}}$$

BENDING MOMENTS.

There is probably no section of the regulations which has been more criticised in the past than that dealing with the bending moments to be provided for, and there was very good reason for the criticisms made, as the question of continuous beams was treated in an indifferent manner, and endeavours made to lay down rules which did not agree with the laws of mechanics, and thus scientific design was hampered and rendered of no account. We are glad to see that in this respect the Council have mended their ways, and there can now be no cause for complaint, inasmuch as they have amended clause 35, which deals with the maximum bending moments due to variations

in the incidence of distributed loads over approximately equal spans, and, furthermore, have introduced a new clause, which states: "Notwithstanding anything in regulation 35, beams may be designed for the exact positive and negative bending moments which will occur at every cross-section, whether all the spans be loaded, or alternate or any of the spans be unloaded." In this last clause, therefore, the designer is *permitted* to design a beam to resist the exact bending moment, and surely the regulations would be very curious if such a condition did not exist, and it rather suggests that the Council are not quite prepared to substantiate the bending moment formulæ as expressed in their own regulations. If such is the case, we would suggest that they are superfluous, and the only rule necessary is that quoted above. The bending moments for slabs will, of course, be calculated as for beams. We notice that the definition of the effective span has been altered to say that it can be considered as the span between the centres of the necessary bearing surfaces, or the clear span plus the effective depth of the beam or slab at the supports, whichever may be the lesser. Various other minor alterations have been made which cannot be mentioned here, but it may be said that it is now stated that the reinforcement shall be carried beyond the points of contraflexure, under any condition of loading, by a length at least equal to half the effective depth of the beam, whereas previously it was only provided to be taken at least as far as the points of contraflexure, but not beyond.

WORKING STRESSES.

The table of working stresses for concrete has been completely revised, and higher values are now permitted, and the increase for richer mixtures has been made more in proportion with actual values, and thus this is an improvement. It is impossible to state all the values here; but, as an example, the direct compression stress under the previous regulations for a 1 : 1 : 2 mixture was only 600 pounds per square inch, whereas it is now given at 750 pounds, and it may be said that other stresses are dealt with in a similar manner. Various other revisions have been made in Part II. which are more or less important, but none of them will be of such interest to the designer as those mentioned.

BEAMS.

Part III., which deals with beams, has been subjected to alterations, and one of the most important of these affects the question of compressive reinforcement. In the previous regulations the stress on the steel could not be taken at more than fifteen times that on the concrete; but this has been revised, to allow the steel to be considered as stressed up to 16,000 pounds per square inch if the compressive resistance of the concrete is not taken into account, subject to the steel being anchored in accordance with certain conditions. This is of great importance, as in the case of shallow beams, where double reinforcement is necessary, the limitation of the stress on the steel to fifteen times that on the concrete involves considerable waste of metal, and often proves prohibitive in cost. The regulations for shear have undergone changes, and a new clause provides that where the vertical shear is taken by the concrete only, the ends of 50 per cent. of the bars of the tensile reinforcement shall be inclined across the neutral plane of the beam, and shall be carried through a depth equal to the arm of the resistance moment, or the whole of the bars shall be carried through to the ends of the beam.

SLABS.

The clause relating to distribution bars has been amplified, and the regulation dealing with the width of slab that can be calculated as acting as the flange of a tee beam has been amended. In the latter case the width has been increased to the distance between the centres of the ribs of the tee beams, or twelve times the thickness of the slab, provided such width does not exceed a fourth of the effective span of the tee beam. In addition, a clause has been added which deals with L-beams. Various other amendments have been made, and amongst these is the alteration of the regulation limiting the effective depth of slabs, which was originally stated to be not less than 4 in., and is now given as 3 in.

PILLARS.

Pillars are dealt with in Part IV. of the regulations, and here, again, many alterations have been made. One, the first that is met with, is that relating to the joints in the vertical reinforcement. As now stated, the clause provides that there shall be an overlap at least equal to twenty-four times the diameter of the upper bar, and this is far more reasonable than the original stipulations. It may be said that some parts of this section have been revised to such an extent that they are no longer recognisable, and although, in some cases, the revision is a question of form only, in others the revisions materially affect the design, and, generally speaking, the regulations as now put forward certainly allow the designer more latitude, and are therefore welcome, there being no necessity to have rules which do not allow of the designer's skill being utilised. For details of the actual alterations that have been made we would commend our readers to the two sets of regulations, as space will not permit us to go into them fully.

WALLS.

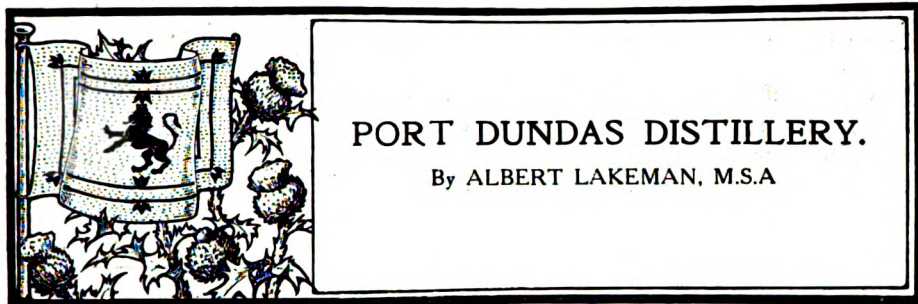
The chief alteration in this section applies to the requirements for hollow blocks, and these are given more in detail, and allow thinner sides to be used than formerly.

The sections dealing with foundations and protection have undergone no change, and in materials and testing the chief alteration is that dealing with the question of deflection, this having been put on a more satisfactory basis. Some small alterations have been made in the section devoted to Workmanship, these relating to concrete, or mortar, which has been frozen, which is now prohibited from use, and to the fixing of wood in concrete for fixing purposes.

CONCLUSION.

Generally speaking, the regulations as now revised appear to be reasonable and efficient, and it is to be hoped that the consent of the Local Government Board will be obtained, and the rules brought into force as quickly as possible, and should any modifications be found necessary at a future date, when experience has been obtained by their application, there should be no difficulty in making them, after proper consideration has been given by all interested parties.

We have only one stricture again to put on record—namely, in respect of the extraordinary delay *in arriving at the present draft*. The Act was passed in 1909 authorising the issue of these regulations. We are in 1915, and six full years have elapsed where one should have sufficed to arrive at an understanding. The progress of reinforced concrete has been materially hindered in the meantime.



The building here described is particularly interesting as an example of the application of reinforced concrete construction to factory buildings.—ED.

THE important buildings erected at Port Dundas for The Distillers Co., Ltd., from the designs prepared by Messrs. F. Burnet and Boston, architects, of Glasgow, contain many interesting features, and reinforced concrete has been employed for the constructional material throughout. When designed in an efficient and economical manner, as is the case in these structures, reinforced concrete is cheaper as regards initial outlay, and it has the further advantage of being peculiarly adaptable for structures of this type, which essentially have a character quite distinct from the ordinary factory or warehouse type, while fire-resistance, good lasting qualities, and homogeneous construction are of primary importance.

The main new building is the Malt House, which adjoins Harvey Street, this having a length of about 240 ft. and a width of 57 ft. The portion adjoining Vulcan Street is the Draff Store, and this has a length of about 90 ft. and a width of 55 ft. An existing silo building occurs at the end of the Malt House remote from the Draff Store, and the kilns adjoin the new building also at the end on the side remote from Harvey Street.

There are four floors, which give a total height of about 63 ft., the lowest of these occurring partly below the ground level in Harvey Street, and being utilised for the drums and having air passages 3 ft. 6 in. deep under the floor arranged to leave passage for bands under the two rows of drums. The first floor is the "steep" floor, each steep of which gives a load of 30 tons to be carried in addition to the calculated superimposed load of 1 cwt. per ft. super for the remainder of the floor area. Above the steep floor the hoppers are constructed, these being about 18 ft. high and 13 ft. 6 in. square. They are arranged in rows across the width of the building, and the total weight of grain in each hopper when full is $66\frac{1}{4}$ tons. The floor over the

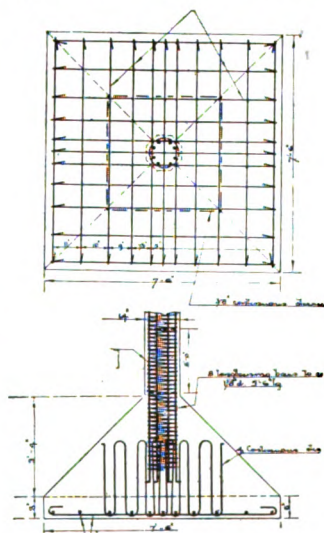


Fig. 1. Foundation Details.
PORT DUNDAS DISTILLERY.

hopper is 9 ft. high, and it is calculated to carry a superimposed load of 1 cwt. per ft. super. It will be seen from this section and particulars that considerable loads had to be carried, and the type of building is not a common one.

The Draff Store is a simple building, consisting of ground, first and second floors, with steel roof trusses over the latter, and with floor construction designed to carry 2 cwts. per foot super in addition to the dead load. With the exception of the steel roof trusses above mentioned, the whole of the work in the new buildings has been executed in reinforced concrete on the *Considère*

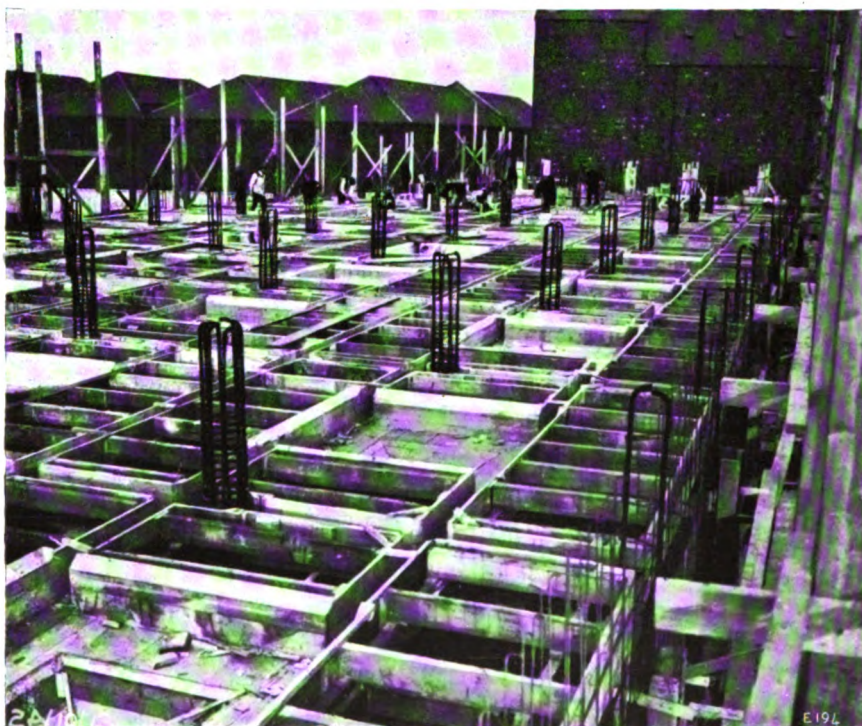


Fig. 2. Work during Construction.
PORT DUNDAS DISTILLERY.

system from designs and details prepared by the *Considère Construction Co.*, of 5, Victoria Street, Westminster.

There are four main longitudinal rows of interior columns in the principal new building at the lowest floor level, and these are spaced at 14 ft. 1 in. centres in the longitudinal direction, with a foundation slab carried down 4 ft. below the level of the bottom of the air passages mentioned previously. The columns generally at the lowest level are 14 in. octagonal, with eight lines of vertical reinforcement, and a typical foundation detail is illustrated in *Fig. 1*, this being 7 ft. 6 in. square, with a minimum thickness of 8 in. at the outer edges, increased to 4 ft. at the point of intersection with the column. The reinforcement consists of a lattice of $\frac{3}{4}$ -in. diameter bars on the under surface and four continuous stirrups passing round these bars and carried well up

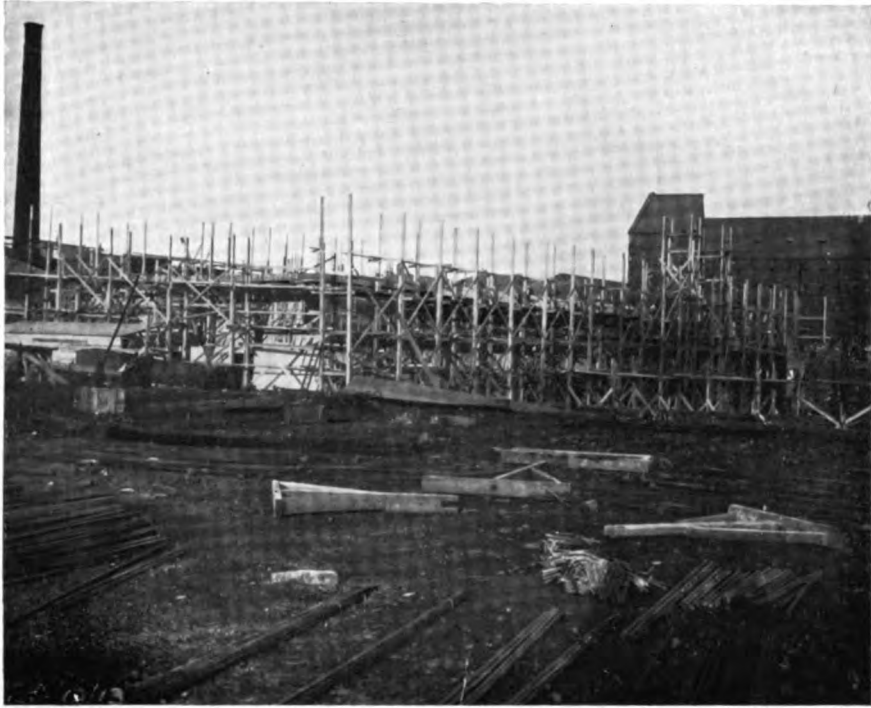


Fig. 3. Building in course of construction.
PORT DUNDAS DISTILLERY.



Fig. 4. PORT DUNDAS DISTILLERY.

into the mass of concrete above, thus anchoring the tensional reinforcement and providing for resistance to shear. In order to enhance the natural strength of the concrete and thus allow the compressive stress to be raised to an economical figure, the vertical reinforcement in the column is encircled with the continuous spiral, in accordance with the Considère system, and the main rods and the spiral are carried well down into the column base. In the case of the columns in the row next the external wall adjoining Harvey Street, the foundation was constructed in conjunction with that under the wall piers—the latter being of reinforced concrete 16 in. square, and having six lines of vertical reinforcement—as the two supporting members are only about 7 ft. 6 in. apart. This foundation is 12 ft. by 8 ft., and it is reinforced to act as a beam between the two columns, with the side portions as cantilevers from either side of this beam, thus ensuring a rigid foundation capable of distributing the load uniformly over the soil. The small columns supporting the floor over the central air passage are probably some of the smallest reinforced columns in existence, as they have a total length from the floor level to the underside of the foundation of 4 ft., and they are 5 in. square with four vertical bars, and a base only 12 in. square. The floor over this air passage was constructed in this manner without cross-beams, but with one central longitudinal beam carried by these small columns, in order that the resistance to the flow of air should be reduced to a minimum.

Owing to the drum floor coming partly below the level of Harvey Street, it was necessary to construct retaining walls for a varying height, according to the fall of the street, the maximum being about 17 ft. As the external walls are of reinforced concrete, with weight-carrying piers at 14 ft. 1 in. centres—

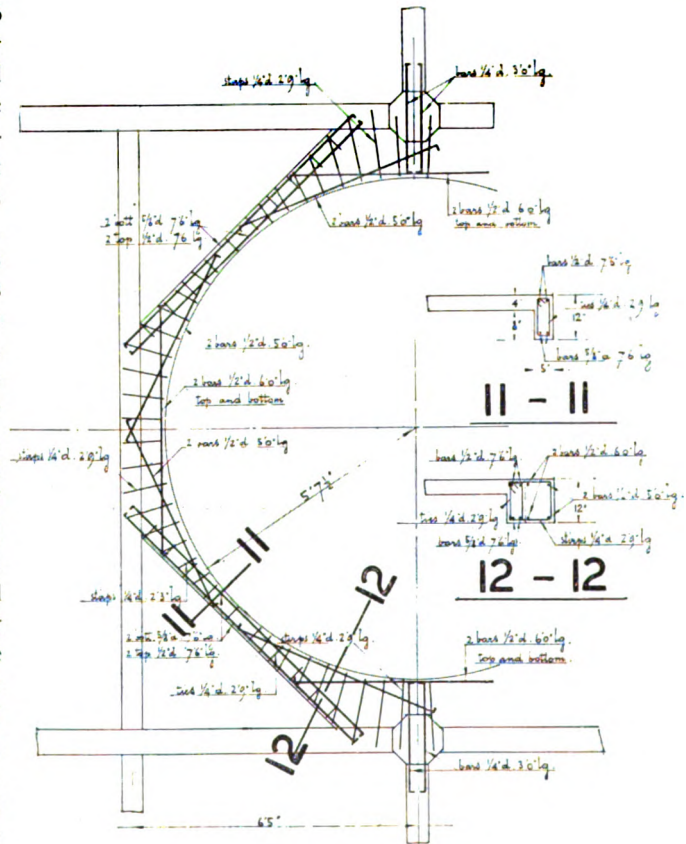


Fig. 5. Beam details to Steep Floors.
PORT DUNDAS DISTILLERY.

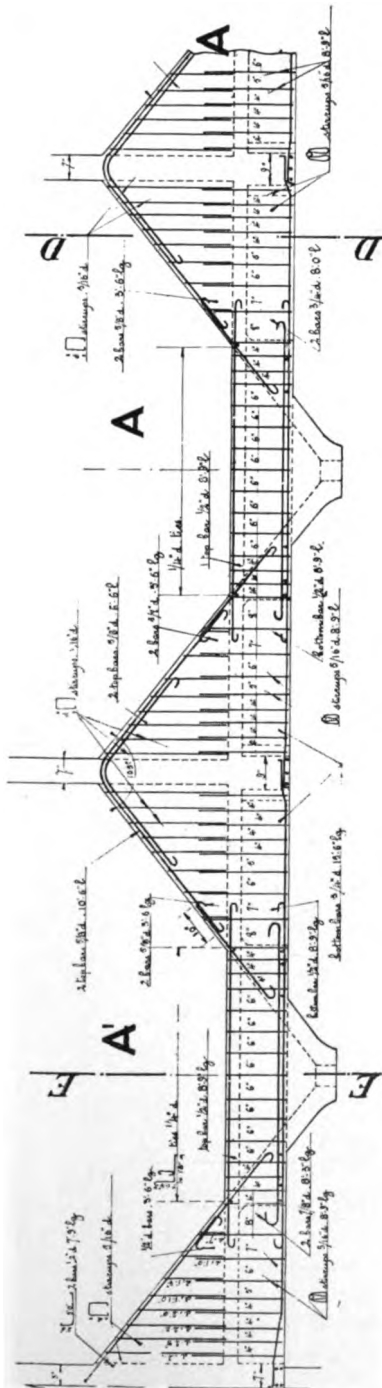


Fig. 6. Section of Hopper Floor.
PORT DUNDAS DISTILLERY.

that is, in line with the internal columns—the portions below the ground are designed to act as vertical slabs, which bring the pressure on to intermediate horizontal beams, and the latter, in turn, transmit the pressure to the main piers; thus the method is similar to that adopted in floor construction when slabs, secondary beams, and main beams are used. The horizontal beams are generally 17 in. by 6 in., and spaced at distances apart varying from 3 ft. 6 in. to 6 ft. 6 in., while the slabs are 5 in. thick, and reinforced with vertical and horizontal bars in the inner surface.

The design and lay-out of the beams and slabs forming the steep floor is obviously governed to a great extent by the size and disposition of the steeps, which are placed in two longitudinal rows. The columns from the floor below are continued up through this floor, and main beams, having a span of 14 ft., are employed, with secondary beams at about 7 ft. centres.

The slabs generally are 4 in. thick, with $\frac{3}{8}$ -in. diameter bars at 5-in. centres and distribution bars at right angles to the main reinforcement, spaced at 12 in. centres. The secondary beams are 8 in. wide and 18 in. deep, including the slab, with two bars as tensional reinforcement in the lower surface and three continuity bars in the upper surface where passing over main beams and columns. The main beams are 9 in. wide and 21 in. deep, with four bars in the lower surface at the centre and three continuity bars in the upper surface where passing over the columns. Where the steeps occur it was necessary to leave a circular opening in the floor about 11 ft. 3 in. in diameter, and this was formed by beams 12 in. deep, which cut off the angles between the main and secondary beams, thus giving an octagonal trimming, that was brought to

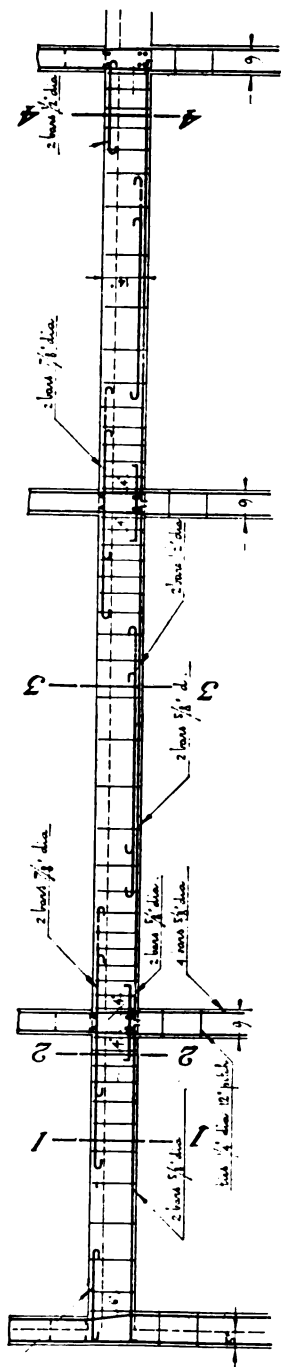


Fig. 7. Beam to Floor over Hoppers.
PORT DUNDAS DISTILLERY.

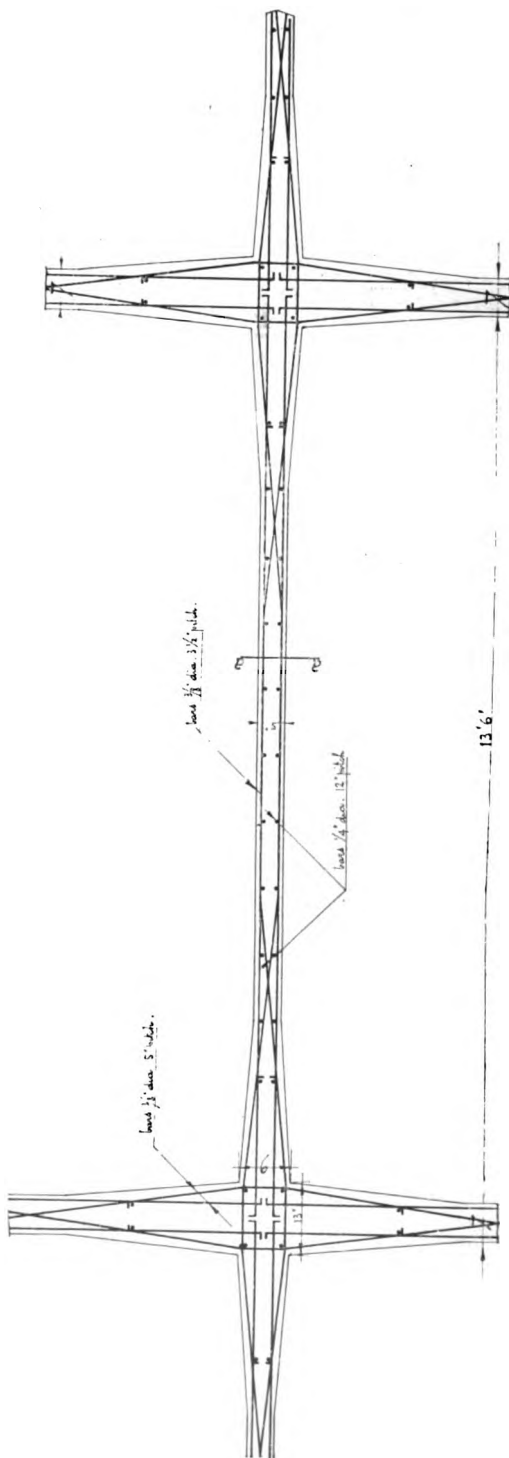


Fig. 8. Wall to Hoppers.
PORT DUNDAS DISTILLERY.



Fig. 9. Interior View of New Building.
PORT DUNDAS DISTILLERY.

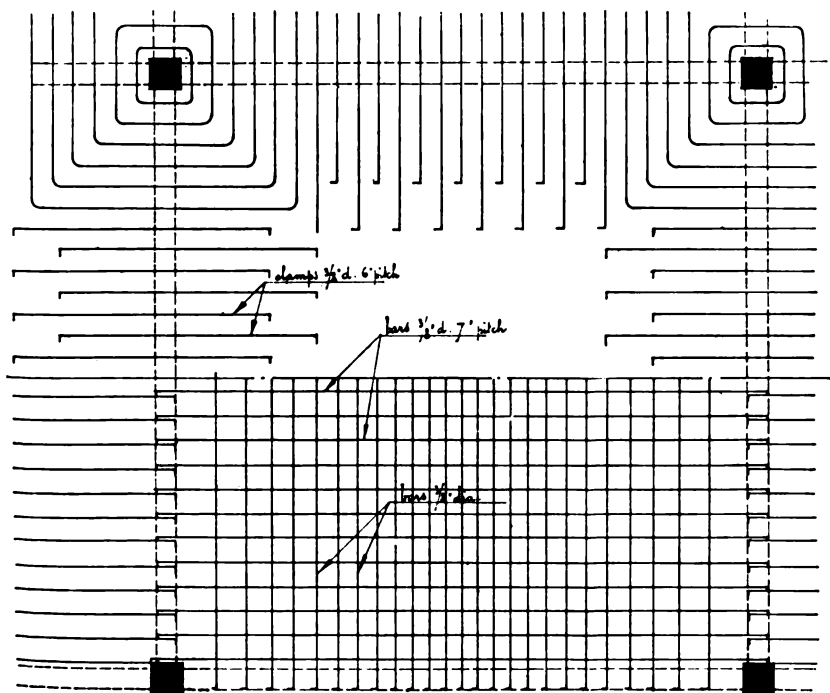
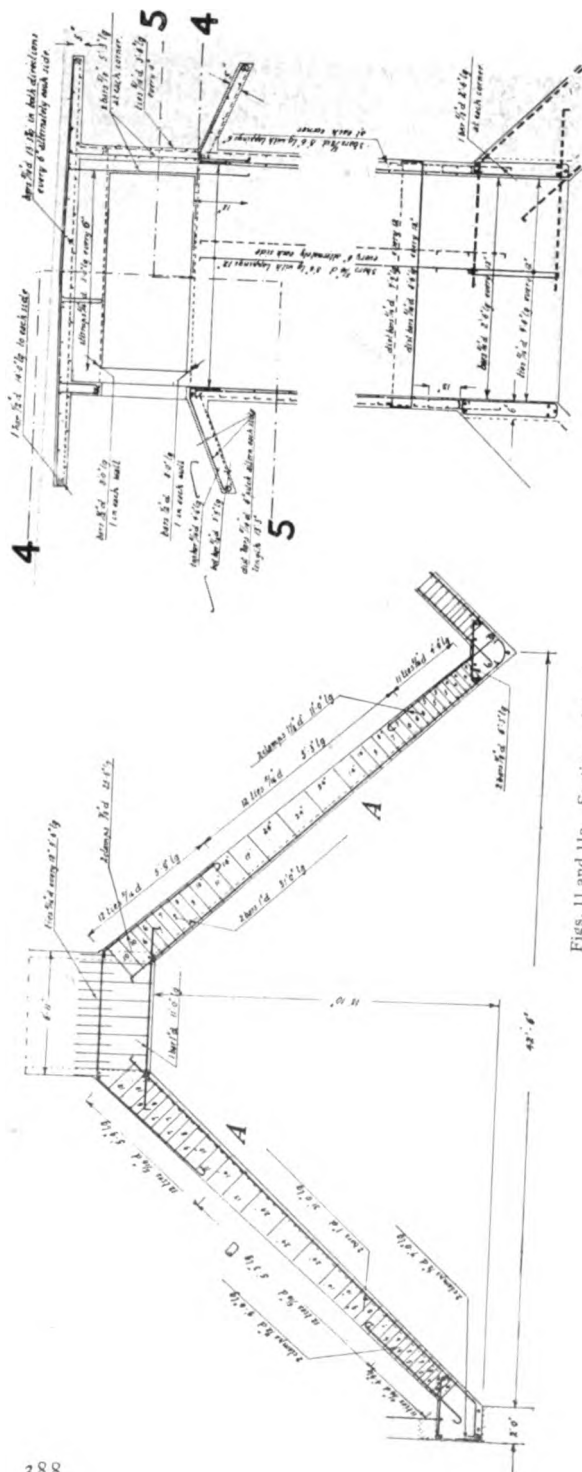


Fig. 10. Square Panel over Hopper Floors.
PORT DUNDAS DISTILLERY.



Figs. 11 and 11a. Section and details, Kiln Roofs.
PORT DUNDAS DISTILLERY.

the finished shape by providing additional concrete to the beams, which made them curved on the side next to the opening, as shown on the detail in Fig. 5. The octagonal columns are carried up to the level of the hopper floor without reduction in size, but above this point they are reduced to 9 in. square, with four vertical bars, and some are stopped entirely. Those that are continued through the hopper floor are again reduced to 8 in. square at the top floor level, and a similar size is employed for those columns which occur in this top floor only. A detail of the hopper walls is shown in Fig. 8, this being 13 ft. 6 in. long, with a minimum thickness of 5 in. for the central 8 ft. of length, and being splayed at each end to give a maximum thickness of 9 in. at the point of intersection with the adjoining walls. The transverse walls are of a similar type, but have a minimum thickness of 7 in. and a maximum of 9 in. A detail of the construction at the bottom of the hoppers is shown in Fig. 13, the sloping concrete having a thickness of 5 in. leading down to the mouth

of the hopper, which is 6 in. square. A wrought iron ring, $1\frac{1}{8}$ in. diameter, is placed round the opening, and the main reinforcing rods are passed round this. The general arrangement of the beams around the sloping portion, and the disposition of the bars, is shown in the details. The plan of the floor panel shows the arrangement of the reinforcement in the floor over the hoppers, and a typical beam detail is illustrated in *Figs. 7 and 10*, this being 14 in. deep and 6 in. wide, with the bars arranged as shown on the drawing.



Fig. 12. Kiln Roofs.
PORT DUNDAS DISTILLERY.

The roof over the malt-house is arranged with four rows of beams, placed longitudinally, these being 16 in. by 6 in., and having a span of 14 ft., and no transverse beams, but 4-in. slabs are provided, with additional reinforcement transversely between the columns, to cut the longitudinal slabs up into panels about 11 ft. by 14 ft. These slabs are constructed with a slope from the centre to the side walls, giving a fall of 12 in. to the roof, and to allow this the longitudinal beams are at different levels, as required. The side walls are formed with parapets 3 ft. high above the roof, and these parapets, and the wall below, are in reinforced concrete 5 in. thick, with horizontal and vertical bars in both surfaces.

The draff store is constructed with three longitudinal rows of interior columns arranged to divide the floors up into panels about 13 ft. square,

and these columns are 11 in. square on the ground floor, reducing to 9 in. on the first floor and 8 in. on the second floor. The type of foundation employed is different to that used in the malt-house, consisting, in this instance, of a square block of reinforced concrete, with a length of side of 4 ft. or 5 ft., according to the load, and a uniform thickness of 18 in. or 2 ft. The reinforcement consists of bars in both upper and lower surface, and in both directions, and continuous stirrups are provided to connect both sets of bars and resist the shear. The floor slabs to the first and second floors are 5 in. thick, carried by

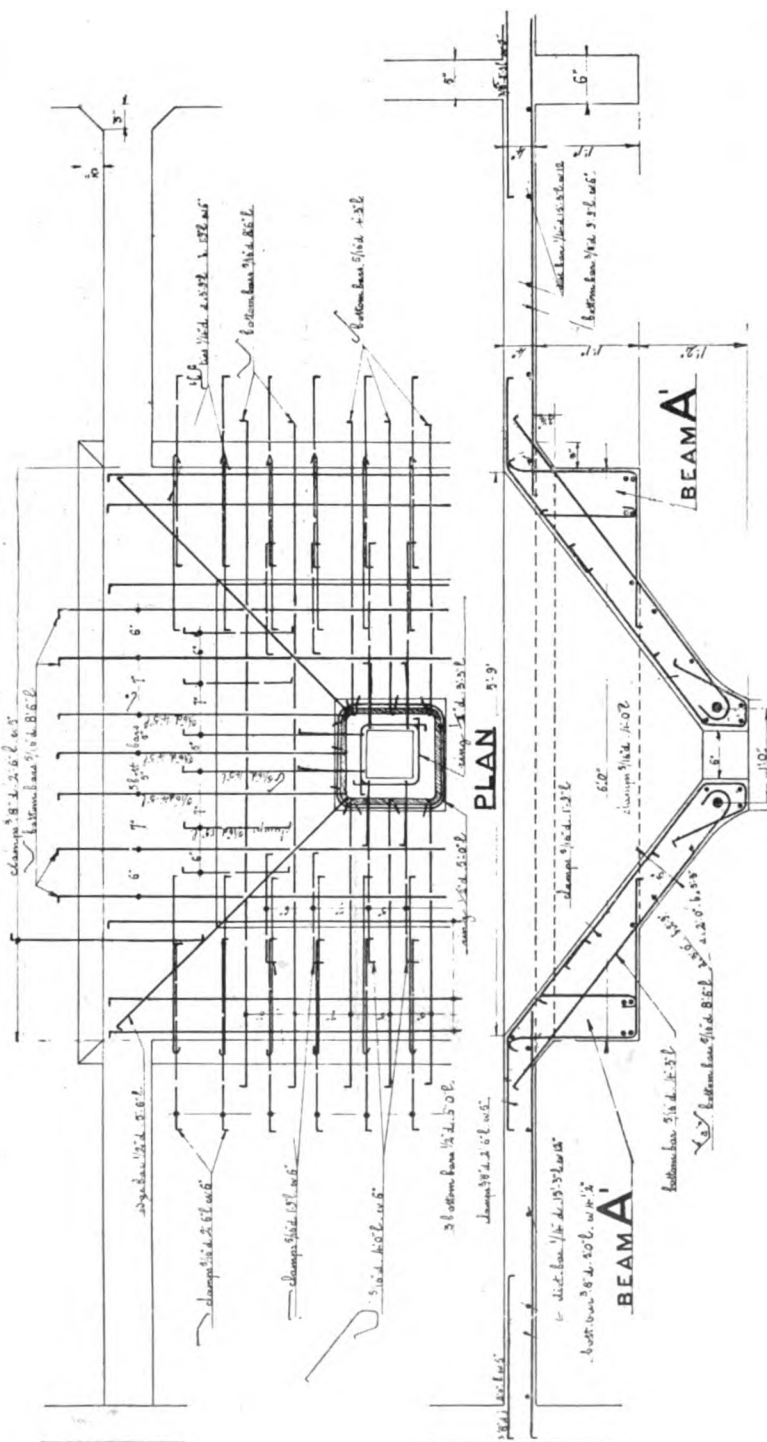


Fig. 13. Details of bottom of Hoppers.
PORT DUNAS DISTILLERY.

beams 20 in. deep and 6 in. wide, which are supported directly by the columns. These beams are reinforced with four bars in the lower surface at the centre of the span, and two of these only are continued through to the ends, while three bars are provided over the columns in the upper surface to give continuity.

The roof is formed with 4-in. slabs laid to a slope, and carried by beams 14 in. deep and 5 in. wide. A chamber, 12 ft. high above the roof, is constructed next the malt-house, with reinforced concrete walls 5 in. thick, while the external walls of the draff store generally are built of brickwork.

Various other work was executed in reinforced concrete in addition to the malt-house and draff store, as, for example, the beams, columns, and hoppers in the grist house, this structure being about 82 ft. by 57 ft. 6 in. In this building

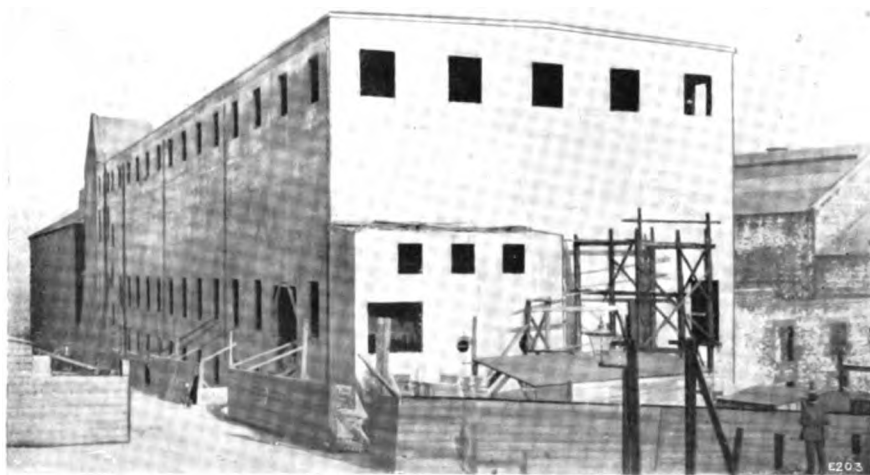


Fig. 14. General View of New Building.

PORT DUNDAS DISTILLERY.

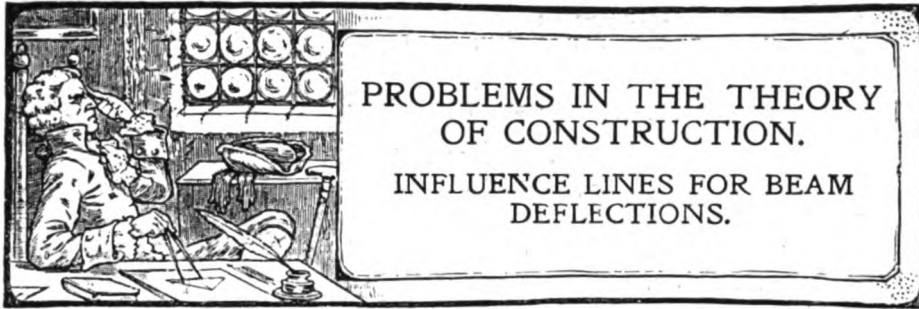
there are some 17-in. octagonal columns, which carry a load of about 260 tons, and these are reinforced with twelve vertical bars, and have a foundation slab 8 ft. 6 in. square. There are also sixteen small hoppers about 8 ft. square, and three large hoppers about 17 ft. square, all constructed in reinforced concrete, in addition to the floor construction at the first floor level.

The new buildings necessitated certain alteration to the kilns and the formation of a new roof, and this was carried out in reinforced concrete. There are four roofs, each covering a space about 45 ft. square, and these are sloped up with a rise of about 19 ft., with a central shaft 6 ft. 3 in. square at the apex, about 25 ft. high. The roof is formed with sloping beams and hips, with 4-in. slabs between, and the vertical shafts have 4-in. reinforced concrete sides. Fig. 11 shows a detail of the sloping roof beams, which are 6 in. wide, and the method of tying the beams at the top will be seen, while the detail

in *Fig. 11a* illustrates the reinforced concrete to the sides and top of the vertical shafts mentioned above. A photographic view of these kiln roofs is given in *Fig. 12*.

There are many interesting features in the new work which it is impossible to describe here; but it may be said that the whole scheme is an excellent example of scientific design and construction, and it is a worthy addition to the works in reinforced concrete.

The contractors for the work were Messrs. Melville, Dundas, and Whitson, of Glasgow.



By EWART S. ANDREWS, B.Sc.Eng., M.C.I.

The following article will be of interest to engineers and others who have to deal with these questions.—ED.

THE use of the influence-line method of dealing with problems in structural design is very useful in finding the deflections in simply-supported beams due to an irregular system of loading. The influence line for the deflection at any point, say P , Fig. 1, of a beam will be a line such that its ordinate at some other point Q will give the deflection $\delta_{P,Q}$ at P due to a unit load placed at Q ; it is a picture of the way in which the deflection at P varies when the unit load moves from one point to another on the beam.

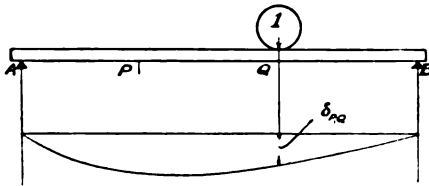


FIG. 1.

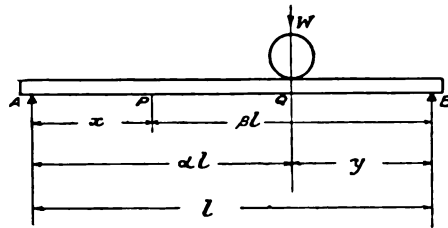


FIG. 2.

Maxwell's Law of Reciprocal Deflections.—We will next prove the above important law, which states that “*The deflection at P for a unit load acting at any other point Q on a simply-supported beam is equal to the deflection at Q for a unit load acting at P .*”

To prove this law we will take a simply-supported beam AB , Fig. 2, of span l , and consider the expression for the deflection at P due to a load W at Q . This can be shown to be equal to:—

$$\delta_P = \frac{W}{6EI} \left\{ (1-a)x^3 - a(1-a)(2-a)l^2x \right\}^* \quad (1)$$

$$\text{i.e., } \frac{6EI\delta_P}{W} = (1-a)x \left\{ x^2 - a(2-a)l^2 \right\} \quad (2)$$

If the load were at P , and we put β for a and y for x in the above equation, we shall get the deflection at Q given by:—

$$\frac{6EI\delta_Q}{W} = (1-\beta)y \left\{ y^2 - \beta(2-\beta)l^2 \right\} \quad (3)$$

*See, for instance, the Author's “Theory and Design of Structures,” p. 218, equation 10.

We have now to prove that the results of equations (2) and (3) are the same. To prove this we note that $x = l - \beta l = l(1 - \beta)$ and $\beta l = l - x$; also

$$y = l - al = l(1 - a)$$

$$\therefore (1 - \beta) y = \frac{x}{l} \cdot l(1 - a) = x(1 - a) \quad (4)$$

$$y^2 = l^2(1 - a)^2 = l^2(1 - 2a + a^2) \quad (5)$$

$$\beta(2 - \beta)l^2 = \beta l(2 - \beta)l$$

We have shown above that $\beta l = (l - x)$; further

$$(2 - \beta)l = (1 + 1 - \beta)l = l + (1 - \beta)l = (l + x)$$

$$\therefore \beta(2 - \beta)l^2 = (l - x)(l + x) = l^2 - x^2 \quad (6)$$

Putting results (4), (5) and (6) into our equation (3) we get

$$\begin{aligned} \frac{6EI\delta_p}{W} &= x(1 - a) \left\{ l^2(1 - 2a + a^2) - (l^2 - x^2) \right\} \\ &= x(1 - a) \left\{ x^2 - (2a - a^2)l^2 \right\} \\ &= x(1 - a) \left\{ x^2 - a(2 - a)l^2 \right\} \end{aligned} \quad (7)$$

This is exactly the same as the result of equation (2), so that we have $\delta_p = \delta_o$, thus proving the law.

Taking $W = 1$, we may write equation (2) as

$$EI\delta_p = \frac{(1 - a)x}{6} \left\{ x^2 - a(2 - a)l^2 \right\} \quad (8)$$

Then the curve drawn for this equation for any given value of x , or for various values of a , gives us an influence line for the deflection at the given point, from which the deflection due to a complicated system of loading may be readily obtained in the manner next to be described.

Case of Deflection at the Centre.—In most cases that arise in practice the deflection at the centre is of greater importance than at any other point.

$\therefore$ Putting $x = \frac{l}{2}$ we have

$$\begin{aligned} EI\delta_c &= \frac{(1 - a)l}{12} \left\{ \frac{l^2}{4} - a(2 - a)l^2 \right\} \\ &= \frac{l^3}{12} (1 - a) \left\{ \frac{1}{4} - a(2 - a) \right\} \\ \therefore \frac{EI\delta_p}{l^3} &= \frac{(1 - a)}{12} \left\{ a(2 - a) - \frac{1}{4} \right\} \end{aligned} \quad (9)$$

[We have divided through by -1 to get rid of the minus sign which arises in the formula and merely indicates that the deflection is downward.]

This formula gives the following coefficients from which the influence line can be drawn, as shown in *Fig. 3*.

Values of a	Deflection Coefficient	Values of a	Deflection Coefficient
.50	.0208	.80	.0118
.55	.0205	.85	.0091
.60	.0197	.90	.0062
.65	.0183	.95	.0031
.70	.0165	1.00	0
.75	.0143		

Since al must be greater than x in Fig. 1, we only tabulate as shown; for lower values we work from the other end and, from considerations of symmetry, the curve will be the same on each side.

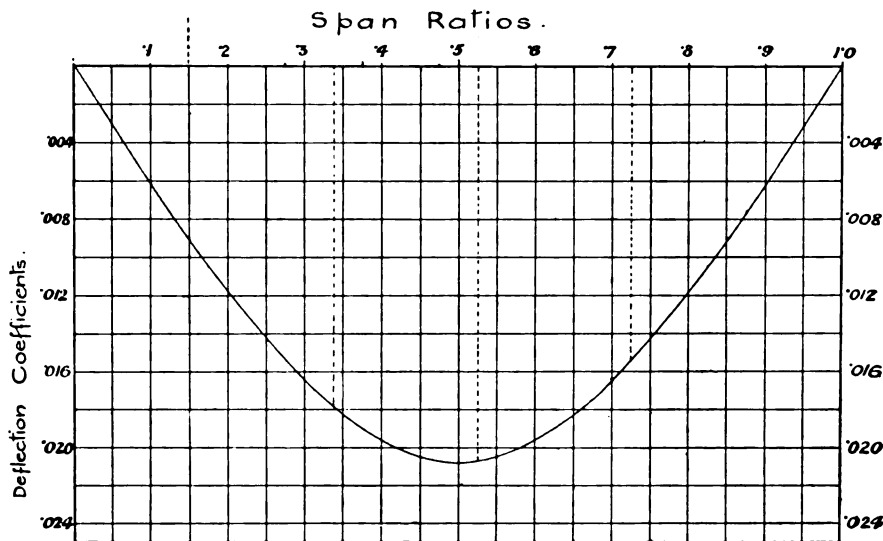


FIG. 3. DEFLECTION INFLUENCE LINE FOR CENTRE OF SPAN.

Plotting these results upon a unit span we get the influence line shown in Fig. 3; to find the deflection at the centre of a beam, due to any given loading, we then place the given loading on the unit span and multiply each load by the ordinate of the curve. The results are then added together, and if this final sum is z , we have the deflection δ_c at the centre given by the relation.

$$\frac{EI\delta_c}{l^3} = z$$

$$\text{i.e., } \delta_c = \frac{zl^3}{EI} \quad (10)$$

This process will probably be more easily followed by the aid of the following numerical example.

Numerical Example.—A girder whose moment of inertia is 9,000 in.⁴ carries the load system shown in Fig. 4, the span being 40 ft. Find the deflection at the centre of the span when the load is in the position shown, E being taken as 13,000 tons per sq. in.

We first place this loading in its proportionate position upon the unit span of Fig. 3, and then draw ordinates to the influence line, as shown in dotted lines; then, multiplying each ordinate by its load and adding, we get

$$\begin{aligned} 20 \times '0091 &= '182 \\ 20 \times '0180 &= '360 \\ 20 \times '0207 &= '414 \\ 8 \times '0154 &= '123 \\ \text{Total} = z &= 1'079 \end{aligned}$$

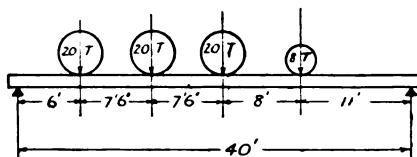


FIG. 4.

$$\therefore \delta_c = \frac{zl^3}{EI} = \frac{1.079 \times 40 \times 40 \times 40 \times 12 \times 12 \times 12}{13,000 \times 9,000}$$

$$= \underline{\underline{1.02 \text{ ins.}}}$$

Deflections at Quarter Span.—If the deflections are required at some other point, say at quarter span, we put the given value of x in equation (8); thus obtaining in the case in which $x = \frac{l}{4}$

$$EI\delta_p = \frac{(1-a)l^3}{24} \left(\frac{l^2}{16} - a(2-a)l^2 \right)$$

Reversing the sign as before, this comes to

$$\frac{EI\delta_p}{l^3} = \frac{(1-a)}{24} \left(a(2-a) - \frac{1}{16} \right) \quad (11)$$

This formula can be used only for values of a greater than $\frac{1}{4}$; for points corresponding to smaller values of a we must work from the other end, putting $x = \frac{3l}{4}$ and $(1-a)$ for a , thus getting

$$EI\delta_p = \frac{3a}{24} l^3 \left(\frac{9l^2}{16} - (1-a^2)l^2 \right)$$

$$\text{or } \frac{EI\delta_p}{l^3} = \frac{a}{8} \left((1-a^2) - \frac{9}{16} \right)$$

$$= \frac{a}{8} \left(\frac{7}{16} - a^2 \right) \quad (12)$$

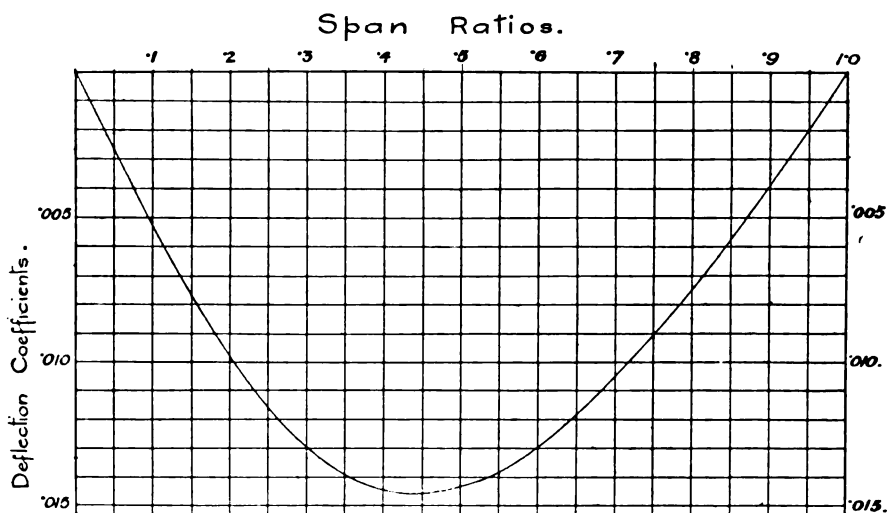


FIG. 5. DEFLECTION INFLUENCE LINE FOR QUARTER SPAN.

These formulæ give the following results :—

Values of α	Deflection Coefficient	Values of α	Deflection Coefficient
0	0	'55	'0139
'05	'00272	'60	'0130
'10	'00534	'65	'0119
'15	'00778	'70	'0106
'20	'00994	'75	'00911
'25	'0117	'80	'00750
'30	'0130	'85	'00572
'35	'0139	'90	'00386
'40	'0144	'95	'00195
'45	'0145	1'00	0
'50	'0143		

Fig. 5 shows the resulting influence line for deflections at one-quarter span, and is used in any particular case in exactly the same manner as described for the deflections at the centre.

In the case in which the load is uniformly distributed over the whole or part of the span, we proceed in the manner common to influence lines by finding the area of the influence-line diagram below the load and multiply by the intensity of the load; this result is used for the value of z in equation (10).



MICROSCOPE AS AN AID IN PROPORTIONING CONCRETE FOR STRENGTH.

**Demonstrating the Influence on Strength of the Ratio of Cement
to Aggregate "Burden."**

By NATHAN C. JOHNSON,
Consulting Engineer, New York City.

Third article of series.—In reprinting these articles we would state that the opinions expressed are those of the author, and the Journal as such does not necessarily associate itself with the conclusions arrived at. The articles are reprinted by the courtesy of the "Engineering Record," U.S.A., and the illustrations have been placed at our disposal by the author.—ED.

In the preceding articles of this series there have been discussed certain defects in commercial concretes, together with some weaknesses, primary and secondary, directly attributable to them. In adducing proofs for these discussions the microscope, in conjunction with the photographic camera, has been the principal reliance. The same means will be employed in the investigations in this present paper, but the manner of their utilisation and the character of the problems attacked will be somewhat different.

MICROSCOPE A HANDY TOOL.

In order that these variations of procedure may be understood, it may be well to emphasise here that the microscope is not so much a delicate, scientific instrument as it is a very handy tool that does not necessarily require a prolonged training for its use. There is no mystery or complication about a field glass. That is an everyday affair, used to magnify a distant object until it becomes sensible to our limited range of vision. Nor is there any mystery about the ordinary magnifying glass, which shows an enlarged image of a small object near at hand. Yet, if an arrangement of lenses similar to this field glass or this magnifying glass is so mounted as to be mechanically convenient, it is invested with the dignity that goes with the name of "microscope"; and, inferentially, its use is prohibited by any save grey-haired scientists. On the contrary, the microscope is a most convenient tool, adapted to a wide range of uses. Nor is it necessarily complex, or delicate, or costly. Even an inexpensive instrument can give very excellent results, and will open the door to a field undreamed of by those who know matter only in the gross.

Micro-photography is merely the using of photography to render permanent the otherwise transient images seen through the microscope. Essentially, it is nothing more than a camera-bellows fitted to the eye end of the microscope; and its employment should be no more complex than the use of a hand camera. If the photo-micrographs used to illustrate these discussions are considered as ordinary pictures taken by a magnifying lens in an ordinary camera, it may aid in an understanding and a proper valuation of them. In cement investigations this conception is nearer the truth than may be imagined, for it is often of the greatest service to take direct photographs of the concretes at from one-half to four powers, without the interposition of a microscope. By this procedure the character of gross sections and the interrelation of large aggregate may be studied, while, by supplementing these photographs with others taken at higher

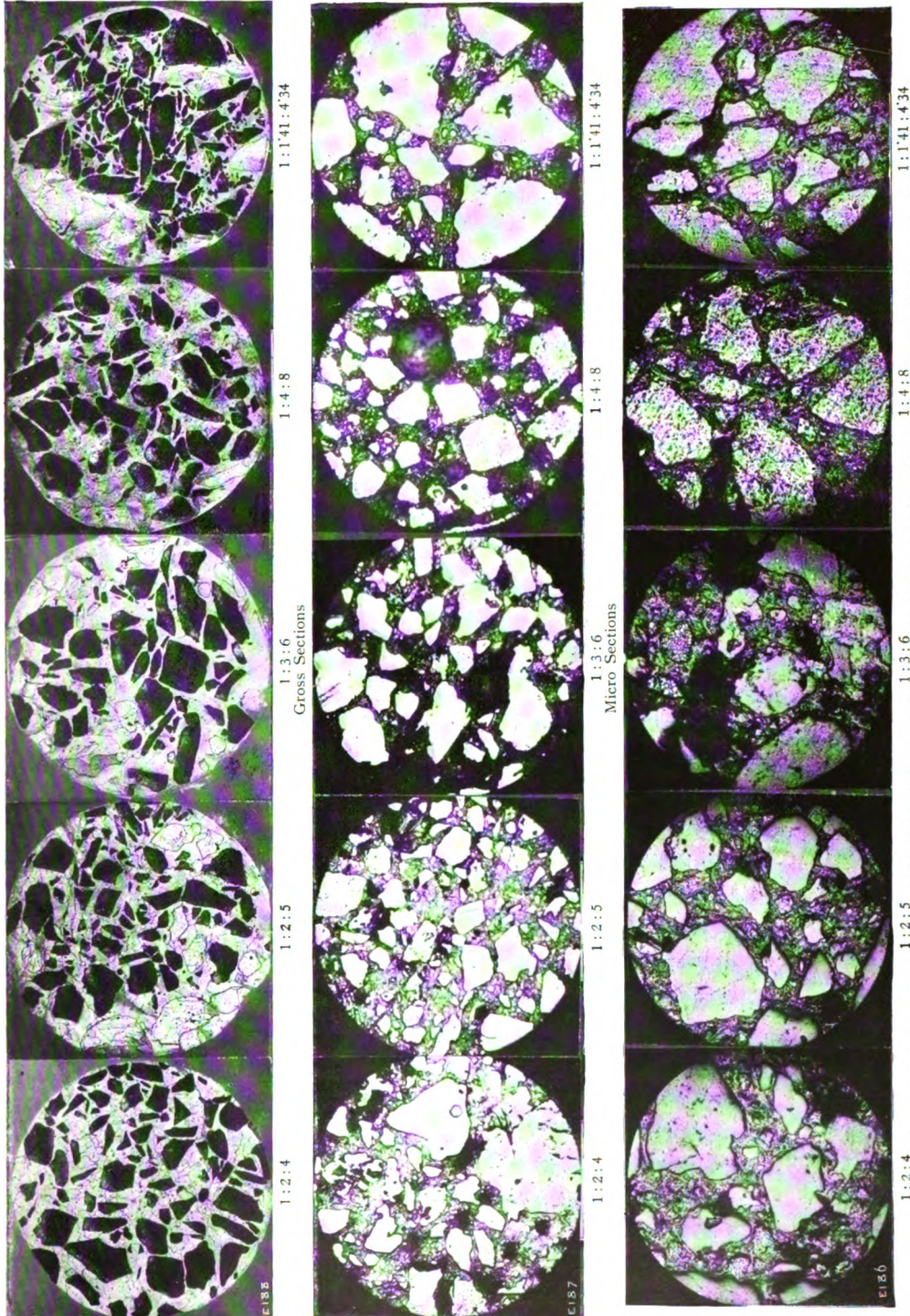


Fig. 15. Gross and Micro Sections of Different Mixtures, to show Character of Matrix as ratio of Cement to Aggregate "Burden" is changed.
THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

powers through the microscope, the entire range from gross to micro-structure is covered, so that true values are established by comparison and erroneous conclusions prevented. This procedure will be followed in obtaining evidence for the discussions of this paper.

Having been chiefly concerned in previous articles with certain defects of concrete, and having learned of some of the consequent ills that concrete is heir to, it becomes of prime importance to inquire as to how those defects may be either remedied or wholly eliminated, so that future construction may be more reliable.

In all probability, those concretes which have so far been examined—all of them commercial or field-mixed concretes, as distinguished from laboratory-mixed concretes—had proportions of cement, sand and stone such as 1:2:4, 1:2½:5, 1:3:6 or 1:4:8, since these are "standard" in field work. Yet, although proportioned and mixed in accordance with accepted practice, the concretes seem to be decidedly faulty.

It would be most interesting to know why this is so. There certainly is no denying the fact. Should the cement be blamed? That, at least, is current practice. Were the proportions wrong? Was there loam in the sand—whatever of good or evil that may really mean? Was the mixer in poor condition? Speculation is easy, but unfruitful of results, unless it should lead to an inquiry as to what the "old reliable," "practical" 1:2:4 or 1:3:6 or any of these arbitrary mixes really are. It is a heretical question, but as it is one of prime importance it may be well to answer it by running the gamut of these proportions in laboratory mixes, testing them in the usual ways, then sectioning and polishing both large and small specimens to determine the internal behaviour and arrangements by means of the photographic camera and the microscope.

PHOTO-MICROGRAPHS OF DIFFERENT MIXES.

In the top row of *Fig. 15* are shown one-fourth-power photographs (taken by direct photography) of five laboratory test cylinders, each 8 in. in diameter by 16 in. long, all made with the same aggregate. Four of these are stereotyped mixes, proportioned 1:2:4, 1:2:5, 1:3:6 and 1:4:8. The fifth is an unrecognised friend, so far as most field work is concerned—a concrete made with the same aggregate as was used in the others, but graded to conform to Fuller's curve of maximum density. (Wm. B. Fuller in "Concrete, Plain and Reinforced," by Taylor and Thompson, p. 183.) Its proportions are 1:1.41:4.34.

Obviously the photographs in the top row have been touched up. The actual photographs of the sections were rather small. These were enlarged 1½ times full size, to a diameter of 12 in. To bring out the various features, red and black drawing inks were resorted to—the black to bring out the stones and outline the boundaries of those dislodged in grinding, and the red to make prominent voids and cracks. These retouched enlargements were then re-photographed in reduced size, so as to render the gross structure of the concretes more readily comparable with the micro-photographs of their several mortars, which latter are placed below. In this way the internal arrangements of these mixes become visible, even to the smallest particles; and by these means the causes of vexatious faults may be determined. When causes are known, remedies usually suggest themselves. With the exceptions of the Fuller's curve and the 1:4:8 concrete, the cylinders that were sectioned and polished had been tested in compression to the yield point. For convenience with the former and because of the weak nature of the latter, the sectioned cylinders of these two were not crushed, compressive strengths being determined on like specimens made at the same time.

With these two exceptions, therefore, it is possible to see how the several concretes bore their load; and how and at what point they failed—a sort of concrete character analysis. The areas covered by stones dislodged in grinding were not blacked in, because the matrix in which the stones were embedded reveals several important matters which it was desired not to obscure.

WHAT THE PHOTOGRAPHS SHOW.

These photographs will repay a very careful study. First of all should be noted the very great number of pores and voids, large and small, not only between the stones, but *especially in those places where stones were bedded*. This is a direct

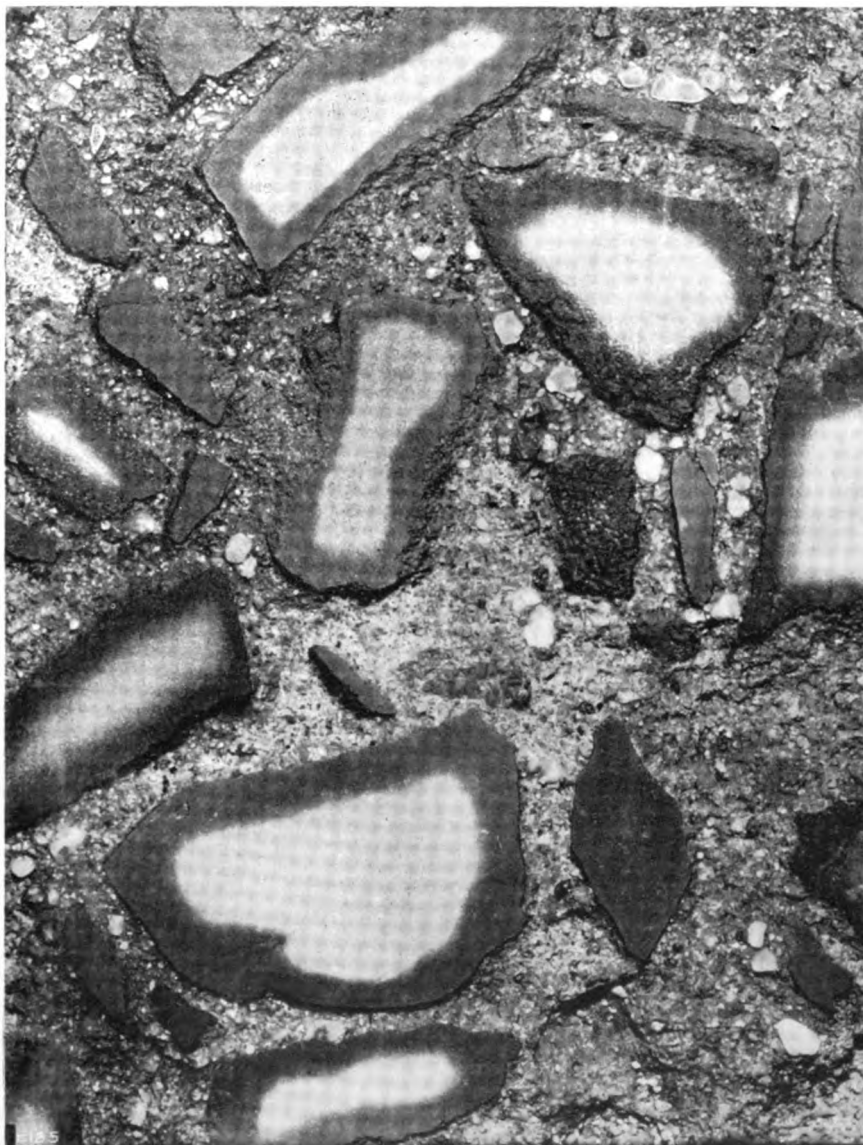


Fig. 16. Weak Character of Matrix of 1:4:8 Mix.

THE MICROSCOPE IN THE STUDY AND INVESTIGATION OF CONCRETE.

confirmation in gross sections of the isolation of small aggregate shown by photomicrographs in Article 1. The bridging of air voids by aggregate is not a theory. It is an actual, provoking fact, and will remain so *unless means are found to prevent adhesion of air to aggregate*. This question will be touched upon more fully in a subse-

quent article. The next feature to which attention is directed is that all of the shear planes due to crushing *run from pore to pore*, whether in the matrix visible between the stones, or in that portion which would be covered had all the aggregate remained in place. This is but one more bit of evidence as to the pernicious effect of air entrained in concrete, as well as additional confirmation of the reasons previously adduced for the necessity of using superior aggregate to produce a very inferior product.

STRENGTH OF THESE MIXES.

Remembering that the greater the proportion of stone in a concrete, the greater its strength and density, it is interesting next to estimate the *strength* of these several mixes, from the appearance of their sections.

Obviously the first of the series, the concrete proportioned according to Fuller's curve, or the 1:1.41:4.34, shows the greatest proportion of stone. It also is the strongest by test, by a few pounds, 2,420 lb. per square inch at 14 days. (This age is too early to give strictly representative crushing strengths.)

Next in order of stone content is 1:2:5, with a strength of 2,280 lb. per square inch, as would be expected from the ratio of stone to sand. The others of the series hold this ratio constant, so that their surfaces show approximately equal stone areas.

RATIO OF CEMENT TO AGGREGATE "BURDEN."

However, the concretes grow progressively weak. If stone content alone were the determining factor, this would not prove true, but the question of ratio of cement to sand plus stone now demands consideration. In this the photo-micrographs of the several mortars, shown in the other two rows of *Fig. 15*, are of considerable assistance.

In any concrete, or in any mortar, the function of the cement is not only to coat the sand and the stone with an adhesive film, but also to fill all voids between the sand particles and the pieces of stone. If the ratio of either or of both of these materials to the cement is increased, or decreased, there is a corresponding change in the burden on the cement, both as a void filler and as a coverer of surfaces. Bearing this in mind, it is no wonder that concretes of different proportions have varying strengths. Since the 1:1.41:4.34 concrete has proved to be the strongest, it may be taken as a basis of comparison. If surfaces of aggregate are assumed as proportional to volumes, the ratio of cement to surface of sand and stone in this concrete is 1:5.75. In the 1:2:5 concrete it is 1:7. In the 1:2:4 concrete it is 1:6; in the 1:3:6 concrete it is 1:9; and in the 1:4:8 concrete it is 1:12. The same ratios should hold approximately true for the relation of cement to voids. These ratios, together with test strengths of the corresponding specimens, appear in the following table:—

RATIO OF STRENGTH TO BURDEN IMPOSED BY AGGREGATE.

Mixture.		Ratio of "burden."		Strength, lb. per sq. in.
Fuller's	...	1:5.75	...	2,420
1:2:4	...	1:6	...	2,200
1:2:5	...	1:7	...	2,280
1:3:6	...	1:9	...	1,012
1:4:8	...	1:12	...	590

MATRIX STRUCTURE.

The photo-micrographs of the several mortars at both 15 and 40 powers confirm what has before been said. The 1:1.41 mortar of the Fuller's-curve concrete shows plenty of cement to coat over and to fill between the sand grains. The 1:2 mortars of the 1:2:5 and the 1:2:4 concretes show the same feature, with possibly something in favour of the Fuller's-curve concrete in the matter of filling voids, due possibly to a greater surface-covering demand on the cement, with resultant thinning of matrix layer. The 1:3 mortar of the 1:3:6 concrete shows a weak, faulty structure, full of holes and with the voids between the sand grains only partially filled with cement,

as is evidenced by the dark spaces in the photographs. The 1 : 4 mortar of the 1 : 4 : 8 concrete exhibits the same character, but to a more marked degree, and with a notably less dispersion of the sand grains relative to one another, due probably to the smaller quantity of intrusive, unhydrated cement. And to confirm our reasoning and our findings thus far, Fig. 16 shows a portion of the 1 : 4 : 8 concrete surface, enlarged to twice natural size, which makes even more evident the weak character of this matrix, with the sand grains loose and ready to fall out. If careless mixing were to fill such a concrete full of air holes, so as still further to weaken it, its fitness for bearing any load beyond its own weight might properly be called in question. This, however, holds true only for concrete made from this particular aggregate. A concrete of the same proportions with different aggregate might be very excellent.

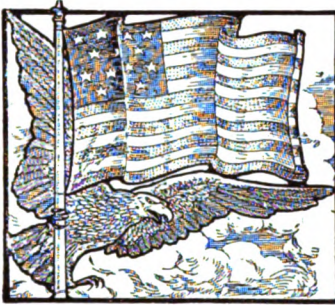
IMPROPER "BURDEN" ON CEMENT.

In view of the foregoing, it is evident that many of the common defects of concrete are due to the imposing of an improper burden on the cement by poor proportioning. Further, arbitrary proportions, or test proportions, that may be suitable for the use of one sand, or for a certain grade of crushed stone, or of bank gravel, may be quite the reverse for a different sand and the same stone, or for the same sand and a different stone, or for a change in both sand and stone. *Each material has surfaces and voids peculiar to itself*, and the quantity of cement must be sufficient to coat these surfaces and to fill these voids, with as little in excess as possible, else the matrix layer will be either insufficient in quantity or else unduly thick, with the production, in either case, of poor concrete. Arbitrary properties are a shibboleth. Sooner or later they will have to be discarded in high-class concrete work; and the sooner this is done the better for the industry. But poor proportioning of aggregates is not alone responsible for the production of poor concrete. In a search for causes as important as that being undertaken, "snap-judgments" are to be carefully avoided. In the next paper of this series certain other factors in the problem will be studied, with special reference to the light thrown upon them by microscopic studies.

SUMMARY.

Summarising the preceding discussion, it should be noted :

1. That the microscope as applied to the study of concrete is not so much a complicated, scientific instrument as a simple, handy tool, capable of rendering visible phenomena and objects beyond the range of unaided vision.
2. That the minute phenomena observed in microscopic study are repeated in the gross; and that gross sections of actual concretes show the characteristic air voids and separation of aggregates that were observed in the mortars previously studied.
3. That the planes of failure in a crushed concrete run from air void to air void, confirming previous reasoning in regard to the weakening effects of such voids, and the necessity of employing aggregate of high strength to bridge these points of weakness.
4. That the prevalence of air voids and fissures in the mortar of gross sections similar in character to the voids and fissures isolating sand grains in the micro-sections previously taken is even greater at surfaces of contact with the large aggregate than in other portions of the matrix.
5. That a concrete having surfaces of aggregate in proper proportion to the cement, to be used both as a void filler and as an adhesive, is the best concrete, both in strength and density.
6. That arbitrary proportions mean little, that proper proportions for certain aggregates do not hold for unlike materials, and that concretes arbitrarily proportioned are usually inferior in density and strength.
7. That a prime factor, though not the only factor, in producing good concrete is *proper proportioning*, and that without this the best materials are rendered valueless.



SOME CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY, U.S.A.

By K. P. BILLNER,

Engineer, Oregon State Highway Commission, Portland, Ore.

The following article is reproduced from the "Engineering News," U. S. A., and also the line blocks, whilst we are indebted to the author for the photographic illustrations.—ED.

THE Columbia Highway, now being constructed, follows the south bank of the Columbia River from Biggs, Ore., through Portland, to Astoria.

The magnificent scenery along the route, particularly between Portland and Hood River, is unsurpassed in this country. Along this portion of the highway are numerous mountain streams and ravines which had to be spanned, and the writer was asked to work out the designs for the bridges. In this work it has been his aim to build structures which would harmonise with the mighty surroundings and at the same time give the factors of safety and low cost their due consideration. No standard types were adopted, but reinforced concrete structures had the preference over steel bridges.

Below are given descriptions of some of the most noteworthy of the structures.

Latourelle Bridge.

The bridge at Latourelle (*Figs. 2 and 3*) is an original type by the writer, although its principles are borrowed from the French expert, the late M. Considère. The principal characteristic of this bridge is its lightness. It is 312 ft. long and 97 ft. high to grade of the roadway. It has a 17-ft. driveway, and the total width, including two cantilever sidewalks and railings, is 25 ft. The concrete above ground amounts to only

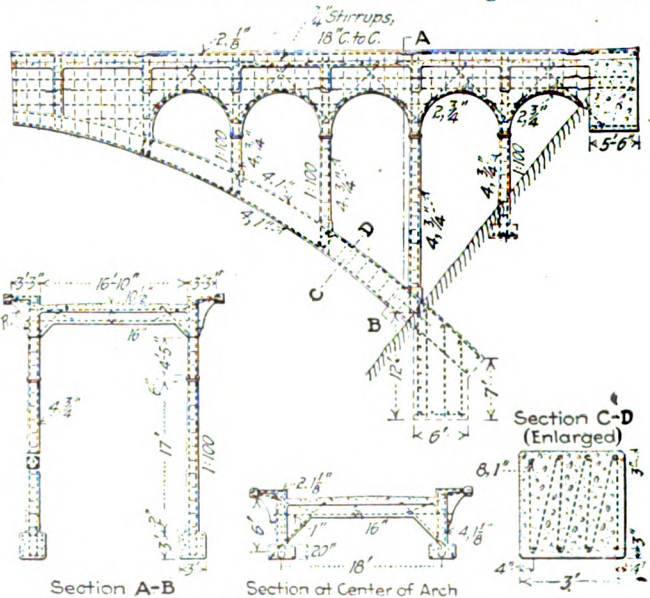


Fig. 1. Details of Shepherds Dell Bridge.

560 cu. yd., making presumably the lightest concrete bridge, relative to its dimensions, in this country. It was desired to erect a light structure for several reasons, among which the difficulty in securing a firm foundation was foremost. The underlying bedrock is covered with a layer of silt and boulders to an



Fig. 2. Bridge at Latourelle.
CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY.

average depth of 25 ft. on the western bank, while on the east side of the creek is a deposit of drift sand, 50 ft. in depth. The cost of building abutments and piers for a heavy type of bridge would have been very high with these conditions of the foundation to contend with.

Substructure.—The abutments, as well as the piers, were founded on bedrock. The west abutment and the two central column bents were placed

directly on bedrock. The east abutment was put on four columns, two 4 ft. square and two 5 ft. square. The average depth of these columns is 45 ft. from the under side of the abutment to the rock. The tops of the 5-ft. columns are connected to the bottoms of the 4-ft. columns by inclined struts, which transmit the thrust from the arches down to the rock foundation.

Superstructure.—Each end of the bridge consists of two girders, which carry a set of columns and struts on which the roadway is erected. This makes a cantilever effect, but the cantilever action which might develop was not considered in the calculations of the stresses in the structure. The central portion of the bridge consists of three 80-ft. arch spans. Two arch ribs carry each span. These ribs are 20 in. square and are reinforced by eight longitudinal 1-in. square bars and hooping of 18-in. diameter and 2-in. pitch. The arches are parabolic in shape. The deck load and any superimposed load are carried to the arch rings on vertical columns placed at 10-ft. centres.

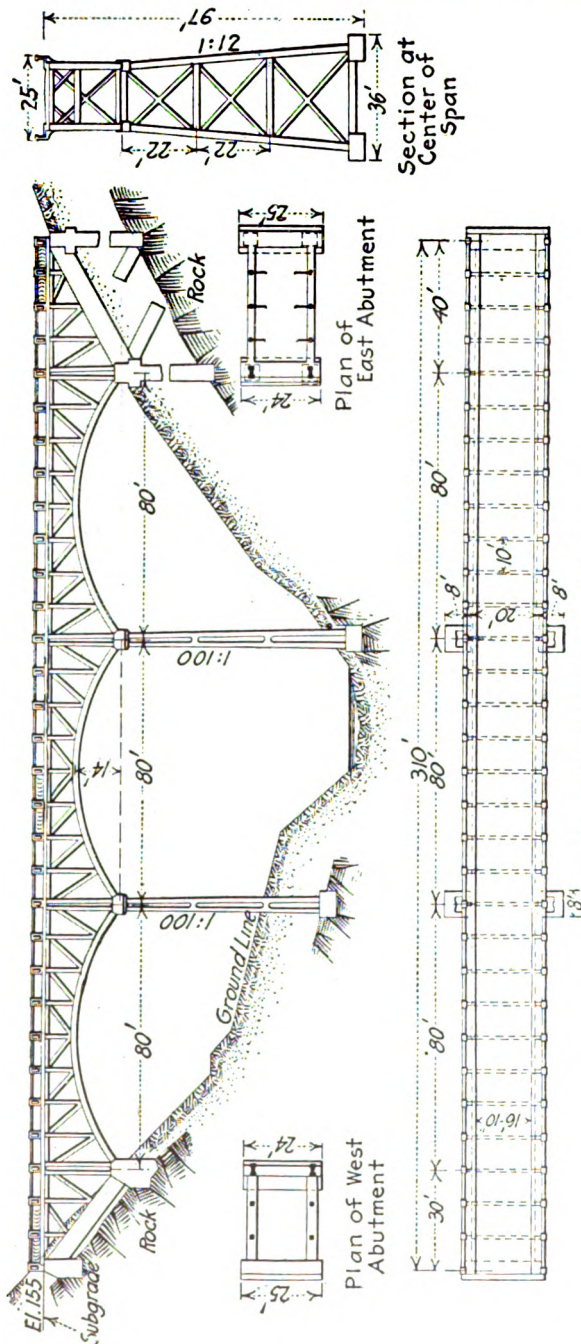


Fig. 3. Details of the Latourelle Bridge.
CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY.

To ensure against bending moments in the arch ribs from partial loading diagonal members are erected between the junction points of the arch ribs and the vertical columns and the deck and the columns. These diagonal

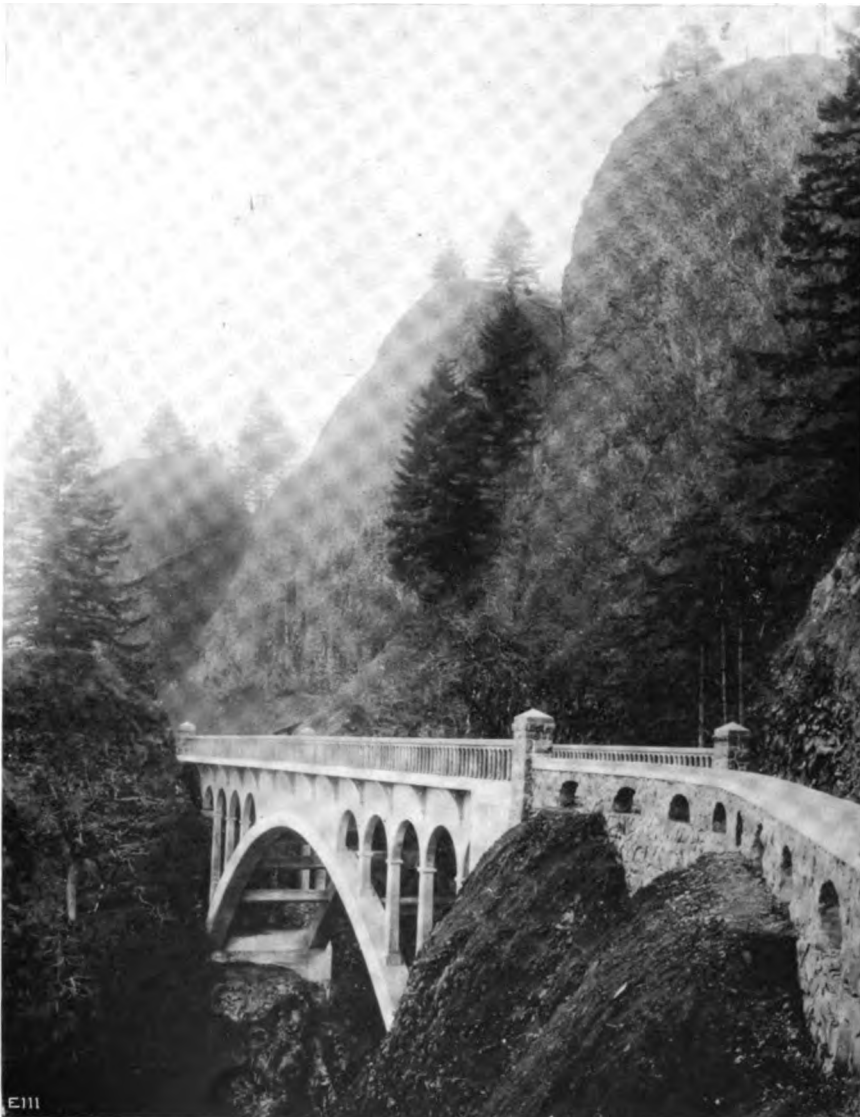


Fig. 4. Bridge at Shepherds Dell.
CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY.

members are subjected to alternately compressive and tensile stresses, and therefore special care had to be employed in the construction of the junction points. The reinforcing bars of the diagonals are hooked around the

longitudinal reinforcement of the arch ribs and also around the longitudinal reinforcement of the girders, which form the edges of the deck. Where it was found impossible to hook the bars to one another as described, special dowels

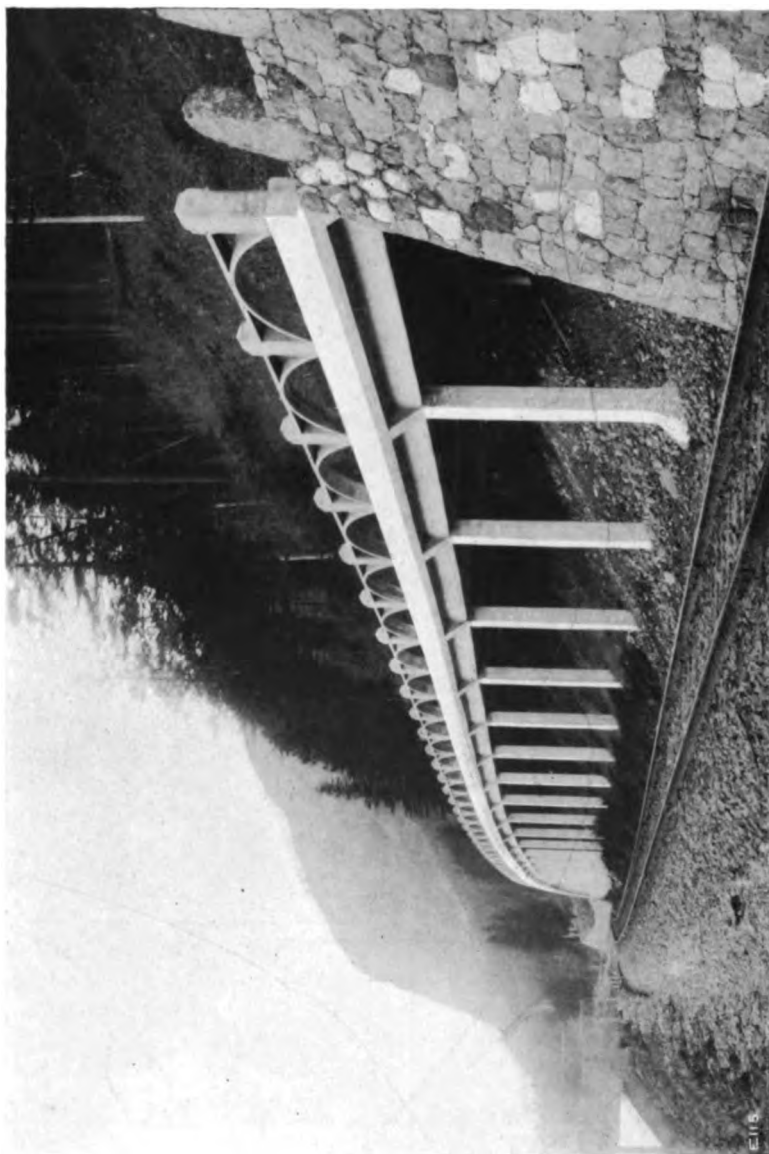


Fig. 5. Viaduct East of Multnomah Falls, 860 ft. long.
CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY.

were inserted in the hooks. The joints received a particularly rigid inspection during the erection of the bridge. The balustrades are made of artificial stone.

Outline of Calculations.—Permissible stresses in the materials were assumed as follows: Concrete in bending, 600 lb. per sq. in. compression; concrete in direct compression, 500 lb. per sq. in.; hooped concrete in the arch ribs,

750 lb. per sq. in.; steel in tension, 16,000 lb. per sq. in.; steel in shearing, 10,000 lb. per sq. in.

Loading.—A uniform load of 100 lb. per sq. ft., a concentrated load of 15 tons, and an impact factor of 25 per cent. were adopted.

The analysis was made for three conditions of loading: First, a uniform load over the entire bridge; second, half the span of one arch loaded; third, two spans were fully loaded, with no load on the third span.

The springing points of the arches were restrained from moving by making the main columns act as beams with double reinforcements.

Construction.—The falsework was erected with a tower at each end of the bridge, and a cableway. All of the forms were built and braced before the concreting of the superstructure commenced. The main columns, which are about 90 ft. high, were poured in sections which were allowed to

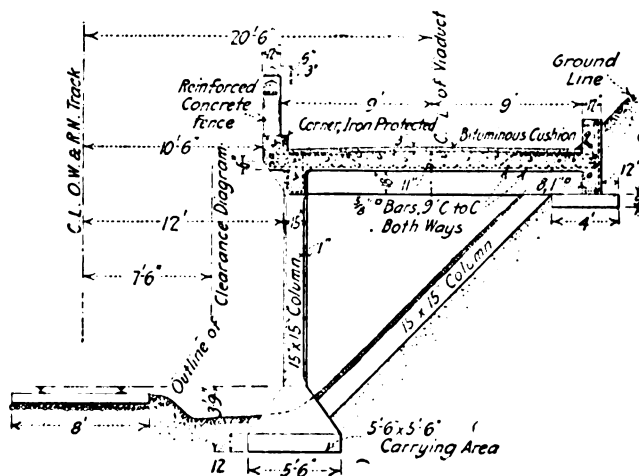


Fig. 6. Section of Multnomah Falls Viaduct.
CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY.

set for a few hours before the next pouring commenced. The arch ribs were poured simultaneously, commencing at the springings and working toward the crowns. The deck, containing 250 cu. yd. of concrete, was poured in a continuous operation lasting 20 hours. The maximum settlement during the pouring of the arches was $\frac{3}{8}$ in. The deck was poured after the arches had set for 12 days. No settlement took place while the deck was being poured or after. The centres were struck six weeks after the pouring of the arches.

Bridge at Shepherds Dell.

This bridge (Figs. 1 and 4) is of a standard type, but has one original feature in the reinforcing of the spandrels. The spandrels are reinforced in such a way as to make them act as girders, and are capable of sustaining the bending moment from the live load over half the span. These spandrels, therefore, will distribute the loading on the arches. The influence of the stiffening of the spandrels, although of great value, has not been considered in the determination of the dimensions of the arch ribs. The clear spans in the Shepherds Dell Bridge are each 100 ft.

Viaducts.

Among problems which arose in the construction of the Columbia Highway were two which necessitated the building of viaducts. A glance at Fig. 6 will be sufficient to explain the situation. The highway here is located

on a steep mountainside, at the foot of which the Oregon-Washington R.R. and Navigation Co.'s line is located. To excavate a 24-ft. roadway out of the hillside would have meant the moving of an enormous quantity of earth with no place to dump it. In order to avoid this, the viaducts were erected.

Fig. 6 shows a cross-section of the viaduct at Multnomah Falls and is



Fig. 7. Highway Bridge, Multnomah Falls.
CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY.

typical of the design of these side-hill viaducts. Essentially the construction is a solid reinforced concrete slab, crowned to the road crown, which rests on transverse floor-beams spanning between columns on the low side and a continuous girder on the high side. These columns are spaced every 20 ft. and foot on prismatic bases. The up-hill girder rests on square concrete plates and the plates and column footings are tied together by reinforced concrete struts

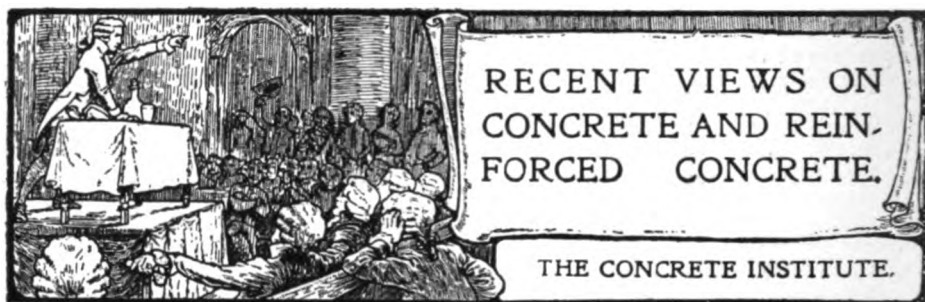
taking the slope and embedded in ground. Longitudinal stiffness is also given by girders at the top of the columns under the railing.

Fig. 5 shows the viaduct from the east side. The peculiar arched railing should be noted. This type of railing was used on a number of the other structures. The total length of these structures is 1,260 ft. Besides eight concrete bridges erected on the highway, there is to be mentioned a footbridge over the lower Multnomah Falls 105 ft. in the air. This bridge is shown in *Fig. 8*. The total length of roadway of these bridges and viaducts is 2,012 ft. The contract price was only \$76,500.

The price per square foot of deck surface varies from \$1.25 for the viaducts to \$3 for the high bridges.



**Fig. 8. Footbridge over Lower Multnomah Falls.
CONCRETE BRIDGES ON THE COLUMBIA HIGHWAY.**



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

THE CONCRETE INSTITUTE. LIME CONCRETE IN THE EAST.

By E. A. W. PHILLIPS, M.Inst.C.E., M.C.I.,
Retired Superintending Engineer, Burma Public Works Department.

The following is an Abstract from a Paper read at the sixty-third ordinary general meeting of the Concrete Institute. An interesting discussion followed, of which we also give a short report. Mr. Chas. F. Marsh, M.Inst.C.E. (Vice-President), was in the chair.

Part I. ORIGINAL PAPER.

(Presented to the Concrete Institute by E. A. W. Phillips on election.)

STONE lime of great purity, and consequently non-hydraulic, is used largely in India and Burma, and engineers have learnt to place considerable confidence in the material. To enable it to set under water it is mixed with "soorkhee," the homra of Egypt.* To the present day engineers in India do not know exactly how much soorkhee is required by each kind of lime, and this ignorance is due to the want of scientific laboratory tests, of the kind so frequently made in Europe. It seems no advantage to send lime and soorkhee to England to be tested, since the difference in climate, the sea voyage, and the lapse of time in transit might vitiate the results.† Conservative Indian opinion, based on long experience, approves of a mixture of a half-part of under-burnt with a half-part of well-burnt soorkhee to one part of slaked lime and one part of sharp, clean sand, all measured in bulk, dry. The materials are thoroughly incorporated and ground in a mortar-mill, either under one wheel pulled round a circular trough by a bullock, or in a pan-machine under a pair of wheels. The mortar should be a thick reddish paste, in which the particles of lime cannot be distinguished by the naked eye. A mortar made in this way sets very well indeed in still water, but it sets comparatively very slowly, and some engineers (the author included) add, when necessary, a proportion of Portland cement to the mixture. The introduction markedly hastens the setting to an extent depending on the proportion of cement to lime. One part cement to one part lime by volume sets apparently as quickly as cement mortar. In the early stages of setting the strength of the concrete is much increased, admitting of early handling and removal of moulding boards. The addition of cement preserves soorkhee mortar in wet foundations from the evils of percolation, and the cement, besides, seems to have a chemical (?) effect on the lime, fixing the particles and aiding in a more solid set.

First-class soorkhee mortar, several centuries old, it has been asserted, exceeds Portland cement mortar, 1 to 3, in strength and impermeability, and is said to be often equal to 1 to 2½. In regard to economy, there is no comparison in Rangoon, cement costing (1900) about two rupees per cu. ft., and lime only three-tenths of a rupee per cu. ft. Further, in comparing the two classes, it has to be remembered that while

* "Soorkhee" in India is finely powdered red brick.

† Bricks and limestone might, however, be sent, if carefully packed in air- and water-tight cases.

native Indian masons are exceedingly clever in dealing with lime, they often make bad mistakes in dealing with Portland cement, and have to be specially taught its use, and carefully watched besides, as they are stupidly stubborn.

Where a mortar mill cannot be obtained, or when the work is too small to justify the expense, stone lime is thoroughly slaked by being steeped for from twenty-four hours to a week or more in fresh water, being kept constantly covered and stirred up at intervals with a rod. The result is a perfectly slaked lime, and a breaking up of all but the very hard particles. The wet lime is first mixed thoroughly with the sand, the particles of the sand tending to still further break up and subdivide the lime. Soorkhee, very finely hand-pounded and screened, is then added in small quantities, and if the operation is properly done, it should be a little difficult to detect that the mortar has not been prepared in a mill. This is the best native Indian practice when a mill cannot be got. Tradition asserts that the stone lime in the Táj Mahál of Agra was steeped in water nearly a year before use.

A suggestion has been made in *Indian Engineering* to apply the term "béton" to mixtures with both cement and lime, reserving the term "concrete" for the usual mixtures. The usual classification for cements and limes is:—

1. Portland cement.
2. Natural cement.
3. Puzzolan cement.
4. Hydraulic lime.
5. Common lime.

With all the above, concretes can be formed, either hydraulic or non-hydraulic, and it is proposed that the term "béton" be reserved for all mixtures of 1, 2, or 3, with either 4 or 5, in forming mortar.

The author has employed the following mixtures in practice, obtaining exceedingly good, reliable, and economical results in all sorts of places, and especially in bad foundations:—

Béton, 1 : 1 : 3 (Equivalent).		Béton, 1 : 2 : 4 (Equivalent).	
1 part cement	} all measured dry.	1 part cement	} all measured dry.
1 part lime		1 part lime	
1 part soorkhee		1 part soorkhee	
1 part sand		3 parts sand	
6 parts aggregate		8 parts aggregate	
Béton, in damp foundations (1 : 3 : 5).			
1 part cement	} all measured dry.		
3 parts lime			
3 parts soorkhee			
5 parts sand			
20 parts aggregate			

In all the above the author employed milled mortar if possible, sifting the cement in last of all, and thoroughly incorporating it with the mortar, as shown by uniformity of change in colour.

In above-ground work, where the free lime will be air-set, the soorkhee might be omitted, in which case two mortars might be made, one lime and sand, and the other cement and sand, in any required proportion, and mixed together.

It is not quite certain, however, that such mixtures will not be set hard even under water. The author has made no experiments to prove this.

In dry climates it is essential to keep hydraulic concrete drenched with water, and covered from the direct rays of the sun. The loss of strength in hastily dried concrete or béton is enormous; and nothing is so easy, in the East, as to destroy good material in this way. The author speaks feelingly from experience, as he has seen so much work, executed with first-class material, in a thoroughly workmanlike manner, ruined because the contractor grudged a trifling expenditure in keeping it wet. Béton especially needs slow and long seasoning, and six months is not too long a period in making blocks of the 1 : 3 : 5 mixture given above.

The worst form of unskilful handling met with in India is over-ramming.

Another error is the laying of concrete too dry. With all deference to received opinion, the author does not believe in the practical value, in India and Burma, of concrete laid nearly dry and thoroughly rammed. The practice may give excellent results in a laboratory, but on work it is bound to fail. The best mixture is a jelly-like mixture, which can be freely shaken by light ramming, all air displaced, and the material

readily and well settled. Heavy ramming is an absolute mistake. The concrete or béton should be laid in small quantities and punned into place, *not* rammed. If too wet, the cement and lime are liable to displacement; if too dry, the ramming must be unduly prolonged, while it is more than probable, in a dry climate, that there will be insufficient water in the body of the concrete or béton for proper hydration. External watering should be only necessary to prevent evaporation.

Part II. EXPERIMENTS.

INTRODUCTION.

Under this heading the author gave an account of various experiments undertaken by himself and others. The Associated Portland Cement Manufacturers (1900) very kindly interested themselves in the matter and gave material assistance, without which the experiments could not have been undertaken.

It seems the Associated Portland Cement Manufacturers have already experimented with mixtures of lime and cement, and of lime, cement, and sand. The results are most interesting, and were communicated to the author in the following letter :—

Extract of Letter dated July 28th, 1914.

"As promised on the telephone, I have been looking up our records in regard to the use of hydrated lime in Portland cement concrete, from which I have been able to summarise the following information :—

EFFECT UPON SETTING TIME.									
						Initial.		Final.	
						Hrs.	Min.	Hrs.	Min.
Neat cement	...	...	...	...	...	1	20	5	30
99.5 parts cement)	...	...	...	...	...	1	15	4	55
0.5 slaked lime)	...	...	...	...	...	1	12	4	52
99.0 parts cement)	...	...	...	...	...	1	5	4	50
1.0 slaked lime)	...	...	...	...	...	1	2	4	47
98.0 parts cement)	...	...	...	...	...	1	5	4	50
2.0 slaked lime)	...	...	...	...	...	1	2	4	47
95.0 parts cement)	...	...	...	...	...	1	2	4	47
5.0 slaked lime)	...	...	...	...	...	1	2	4	47

"From these figures you will see that the addition of hydrated lime appears to have little effect on the setting time of cement (only slightly quickening it), and these results are confirmed by others, which I need not repeat here as the above are fairly representative.

TENSILE STRENGTH OF CEMENT MORTAR WITH VARIOUS PROPORTIONS OF LIME ADDED.

					Lb. per square inch.			
3 parts sand to :—					3 days.	7 days.	28 days.	3 months.
1 of cement	...	...	...	...	175	255	263	328
1 of (0.05 cement)	...	...	...	...	143	198	290	333
0.05 lime)	...	...	...	...	185	213	304	347
1 of (0.1 cement)	...	...	...	...	144	222	330	382
0.1 lime)	...	...	...	...	110	193	241	309
1 of (0.85 cement)	...	...	...	...	83	183	200	238
0.15 lime)	...	...	...	...	110	193	241	309
1 of (0.80 cement)	...	...	...	...	83	183	200	238
0.20 lime)	...	...	...	...	83	183	200	238
1 of (0.70 cement)	...	...	...	...	83	183	200	238
0.30 lime)	...	...	...	...	83	183	200	238

"It will be seen from these figures that the substitution of hydrated lime for more than 15 per cent. of the cement effects an appreciable reduction in the strength of the mortar. The results are not concordant, but, on the whole, the use of smaller percentages seems to be, if anything, beneficial to the strength after 28 days. The addition of hydrated lime made the mortar work 'fatter,' or freer.

"We do not appear to have any comprehensive series of crushing tests on lime-cement mixtures, but it is to be expected that they would, broadly speaking, follow the lines of the tensile tests.

"The experiments which we have from time to time carried out in connection with the waterproofing of Portland cement concrete have shown that the addition of a small quantity of slaked lime is, undoubtedly, beneficial in this connection. I give you here the result of one test as an illustration.

" Amount of water which percolated during 24 hours, under a pressure of 50 lb. to the sq. in., through a slab made with three parts of sand to one of cement.

First slab	...	...	...	...	...	110 c.c.
Second slab	...	...	...	...	...	120 „

115

" Amount of water which percolated during 24 hours, under a pressure of 50 lb. to the sq. in., through a slab made with three parts of sand and one part of a mixture of 0.97 cement and 0.03 slaked lime.

First slab	...	...	...	...	...	4 c.c.
Second slab	...	...	...	...	...	10 „

7

THE EXPERIMENTS.

It was decided to conduct tests on lime and Portland cement concrete at the age of six months. In all, some twelve tests were undertaken with mixtures of varying composition, some of the materials having been kindly placed at the author's disposal by the Associated Portland Cement Manufacturers. It is impossible to give here a full account of these tests, but we give below the final analysis of same.

Part III. CONCLUSIONS.

CONCRETE TESTS.

Two sets of tests, Nos. I. and II., were made with béton (1 : 2 : 4 equivalent); one set, No. III., with béton (1 : 3 : 5); and one set, No. IV., with a scamped mixture supposed to be béton (1 : 3 : 5). To compare this béton with Portland cement concrete, two more tests were made, No. V. (1 : 2 : 4) and No. VI. (1 : 3 : 5).

ANALYSIS OF CONCRETE TESTS.

Test No.	No. of Sam- ples.	Proportions								Average Strength in lb. per sq. in. at 6 months
		Cement	Re- mainder	Cement- ing Material	Re- mainder	Mortar, less Sand	Sand and Shingle	Mortar	Coarse Material	
I.	4	1	13	2	12	3	11	6	8	2,168
II.	4	1	13	2	12	3	11	6	8	1,890
III.	5	1	31	4	28	7	25	12	20	817
IV.	3	1	40	4	37	6	35	13	28	795
V.	2	1	6	1	6	1	6	3	4	3,663
VI.	3	1	8	1	8	1	8	4	5	3,117

MORTAR TESTS.

No. VII.—1 cement, 1 white lime, 1 red soorkhee, 6 sand.

No. VIII.—1 cement, 1 grey lime, 1 red soorkhee, 6 sand.

No. X.—1 cement, 1 white lime, 1 burnt gault, 6 sand.

No. XI.—1 cement, 1 white lime, 1 stock brick, 6 sand.

No. XII.—No cement, 2 white lime, 2 red soorkhee, 3 sand.

No. IX.—1 cement, no lime, no soorkhee, 3 sand.

The idea was to test 1 : 3 mortars of all kinds as far as possible. It will be noticed that the chief difficulty was the classification of soorkhee. Is it part of the sand, or part of the mortar, or should it be left out of consideration?

ANALYSIS OF MORTAR TESTS.

Test No.	No. of Samples.	Proportions								Average Strength in lb. per sq. in. at 6 months
		A			B					
		Cement	White Lime	Grey Lime	Red Soor-khee	Burnt Gault	Stock Brick	Sand	Proportion A : B	
VII.	8	1	1	—	1	—	—	6	2 : 7	645
VIII.	8	1	—	1	1	—	—	6	2 : 7	488
X.	8	1	1	—	—	1	—	6	2 : 7	809
XI.	8	1	1	—	—	—	1	6	2 : 7	663
XII.	5	—	2	—	2	—	—	6	1 : 4	225*
IX.	4	1	—	—	—	—	—	3	1 : 3	3,051

* Assumed strength if experiment had been successful.

REMARKS ON CONCRETE TESTS.

Unfortunately, there was no time to make tests of lime, sand, and soorkhee concrete. The mixture employed would have been 1 lime, 1 soorkhee, 1 sand, and probably 6 of shingle, and the results the writer would have anticipated should have been about 300 to 500 lb. to the sq. in., after six months. Combining the mean of these, say 400 lb., and the average result of No. V., we get a new average of about 2,000 lb. to the sq. in., which agrees pretty well with the average result of the two tests, Nos. I. and II. It seems to the writer that if an extended series of tests were made, covering the range of 1 : 2 : 4 and 1 : 3 : 5 variations of mixtures of cement and lime mortars from cement mortar at one end to lime mortar at the other, it would be easy to plot a curve, which might approximate in the case of concrete to a straight line, to cover every possible variation with sufficient accuracy for all practical purposes. If carefully carried out, these results would undeniably encourage the employment of cement to improve lime concrete at comparatively small cost. Cement has been so employed in India since Portland cement was first manufactured.

REMARKS ON MORTAR TESTS.

The cement and sand test beats all other tests hollow. The writer does not, however, consider that the question has been settled by these experiments. Many more experiments are required before it can be stated with any certainty what effect soorkhee has on English limes.

COMPARISONS OF RESULTS.

Comparisons are difficult on account of this very property of quick setting. Lime and sand mortars do not set thoroughly hard for centuries. Cement and sand probably attain maximum strength in a few years. The results given in this paper show tests at six months. As a matter of fact, a fair comparative test for the béton mixtures would have been at six to ten years, to the six months for Portland cement concrete.

MIXTURES OF CEMENT AND LIME IN THIS COUNTRY.

These mixtures are not so rare as might be believed. The writer knows of at least one builder who employs the mixture, and he also knows of several instances of pointing and walling in private houses, in which a mixture of $\frac{1}{4}$ ton cement to every cu. yd. of lime was used with astonishingly good results at a few months, with the advantage that, although in one case the work was done some ten years ago, the pointing appears to be still hardening. The proportion of cement to lime was about 1 : 4.

THE QUESTION OF COST.

This is, of course, important. The cost of labour in mixing, handling, and applying such mixtures would, with a little experience, be probably the same. If the slower

setting does not affect the cost, or other considerations, it is no disadvantage. Mixing lime with cement gives a fatter mortar, easier to work. In a country where cement is nearly as cheap as lime there seems to be no object in using a mixture, unless it is to get a fatter and possibly more water-tight mortar. Where lime is cheaper than cement it is obvious that in the course of time a mixture of, say, 1 cement, 1 lime, 1 soorkhee, and 6 sand may be better and more reliable than 1 cement to 4 sand. In the former case we get 2 of cementing material to 7 of other material—that is, 1 : 3½; in the latter case 1 : 4. The soorkhee, too, being as finely ground as the cement and lime, aids to fill the interstices of the sand. Clay is recommended for adding to the water-tightness of cement concrete. It is a question whether soorkhee would not be better.

In the East, where carriage is very heavy, where lime is cheap and good, and the native masons understand its use better than the use of cement, such mixtures have a very high value.

It is of the highest importance to the structural engineer and to capital that cement can be utilised as described in these pages. In the absence of reliable tests we are all working in the dark, by rule of thumb.

DISCUSSION.

Mr. D. B. Butler, A.M.Inst.C.E., remarked that the use of soorkhee with lime seemed to be on a par with the use of pozzolana with limes used in Europe. Pozzolana, like soorkhee, was really a sort of half-baked clay, more or less burned, and was used to a large extent in Italy with lime for work under water. As a matter of fact, he believed that something similar was used by Smeaton in the construction of the Eddystone Lighthouse, but that was before Portland cement was known. As to the beneficial effect of adding slaked lime in connection with the waterproofing of cement concrete, the lime closed the pores, made the mortar work fatter, and they got a more dense and less porous material. A substance that would set in three or four years was preferable to one that took 50 years to obtain its greatest strength, therefore that point would have to be considered very carefully in using lime as against Portland cement.

Mr. Morgan E. Yeatman, M.A., thought that the experiment of adding 3 per cent. of lime to prevent percolation might be of value. One often heard the waterproofness of concrete discussed, and its suitability for the side of a reservoir, or keeping out water for a foundation, whether it was necessary to add a waterproof coating of asphalt or something of that sort. Possibly this addition of a small proportion of lime might, in some cases, solve the difficulty. The experiments did not show that the product of the mixed lime and cement was as good as the cement alone would give, but he gathered that cement would be too expensive for use in India. He had heard it said that the use of cement concrete in Egypt had not been found very satisfactory, and the failure had been attributed in some cases to not keeping it wet in a very dry climate. In the dry, hot climates of the East it was very important to keep the concrete well damped for some time after setting.

Mr. A. R. Sage, M.C.I., said it seemed worth while to carry the tests further, both from a physical and an economical point of view, but that required money, which ought to be provided by the Colonial people who were interested in the use of soorkhee. If Portland cement could be manufactured in India, the question of soorkhee did not count for very much.

Mr. W. G. Perkins (District Surveyor for Holborn) said he should be very glad if Mr. Phillips would tell them something about the chemical composition of soorkhee. In London mortar was frequently compounded with the red bricks that were obtained from the demolition of houses from one to two hundred years old. These houses were pulled down and the speculative builder used the old bricks by driving them into a mortar composed of grey lime, 1 to 3 broken brick. As a rule, provided this lime were properly slaked—and it must be properly slaked—they would obtain a very good mortar. These red bricks were very largely used in the time of Queen Anne and the Georges, and perhaps previous to that. Mr. Perkins handed round samples of these materials, which, he said, had been used by speculative builders, and therefore the buildings only just complied with the by-laws. The mixing of lime and cement he thought was a very good practice.

Mr. H. Kempton Dyson (Secretary, Concrete Institute) agreed that the practical tests did not go far enough, and that there was plenty of room for more experimenting. A mixture of cement with lime had often been used by bricklayers in this country to make it work fatter, and it certainly did set well. The tests mentioned in the Paper suggested that there was some marked advantage as regarded strength to be derived from such a mixture, and Mr. Phillips assured them of its economy.

Mr. H. J. Harding had been associated with the late Mr. George Deacon about 20 or 30 years ago in the construction of the Burnley and Kendal Waterworks, for which blue lias lime was used. The principal thing for all lime was that it must be properly slaked and perfectly divided up to the smallest particle. Mr. Deacon carried out the Kendal Waterworks with blue lias lime instead of with Portland cement in the face of great opposition, not only on the part of engineers, but councillors as well. Some years ago, manufacturers in London introduced a ground flagstone mixed with cement, which they sold as cement and which gave a better result than pure Portland cement. He did not think that the addition of clay would increase the strength of concrete. Something harder than clay, if ground as fine, would be much better than clay itself.

Mr. Allan Graham, A.R.I.B.A., observed that, in our Colonies, where Portland cement was difficult to obtain, these experiments would be of considerable value. It certainly was an eye-opener that the addition of a little lime to the concrete, and with so much aggregate added, seemed to reduce the concrete so little from the pure Portland cement concrete. For that reason there was something in this line of research. What had been said about the addition of clay to concrete was the recrudescence of an old heresy when Portland cement was a different material from what it was to-day. It might be used for the sake of economy for mass concrete where they did not require the great strength that reinforced concrete demanded.

Mr. A. Stewart Buckle, M.S.E., who was in Assam at the time of the great earthquake in 1897, gave some of the results of his experiences in railway reconstruction there. He suggested that Mr. Phillips might adopt the accepted spelling of Hindustani words, and call it "surkhi."

The Chairman had great faith in adding lime to concrete to increase its watertightness. He generally used 5 per cent. of hydrated lime, and found it had a considerable effect in reducing the percolation of water; in fact, in some cases it was a good deal better than most of the patent compounds that were sold. He believed it was merely a case of filling up the interstices. In the tests which had been carried out, apparently the grey lime had not been slaked before use, consequently it had yielded extremely bad results. In America there was a very strong section of opinion that very finely divided clay was a good material to add to increase the watertightness of concrete. The difficulty with clay was that they must be very careful what clay they used, and very careful tests ought to be made before they used it. No doubt the late Mr. Deacon used blue lias lime because he was building a big masonry dam, and he wished it to set evenly. His fear was that if he used cement he would get cracks, as one part of the dam might set hard before the other part had settled down; therefore he used, and advocated the use, all through his life, of blue lias lime for masonry dams so that the setting might take longer and the dam settle itself in that particular place without cracking.

MR. PHILLIPS'S REPLY.

In reply, the author explained that he did not recommend the employment of clay to add to the watertightness of concrete; he advocated the use of soorkhee, which he considered would be better. He was in entire agreement with the suggestion that further tests should be made. These might easily continue over a period of four or five years, and even then they would not get to the bottom of the subject. He undertook to contribute to the Transactions of the Institute a considered reply to the various criticisms.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE BRIDGE OVER THE RIVER CALDER, AT STERNE MILLS, NEAR SOWERBY BRIDGE, HALIFAX, YORKSHIRE.

THE illustrations which we are herewith reproducing show a reinforced concrete road bridge constructed on the Coignet system over the River Calder at Sterne Mills, near Halifax. This bridge was constructed to replace an old iron and timber bridge, the object being to supply a better means of conveying across the river heavy traffic.

The bridge is a private one, having a right of way for foot and bridle traffic across it, and it unites some property in the ownership of Mr. A. E. Carroll, for whom the new bridge was constructed.

Messrs. Jackson and Fox, of Halifax, were the architects responsible for the abutment works, which were carried out in masonry with concrete foundations, and for the laying of the roadway, and also for the general supervision of the construction of the reinforced concrete works, the design and plans of which were prepared by Messrs. Edmond Coignet, Ltd., of 20, Victoria Street, Westminster, who were the engineers responsible for the reinforced concrete work.

Owing to the soft nature of the foundations encountered at the abutments, it was found necessary to keep as much as possible of the weight and loads of the main beams from bearing upon the masonry of the abutments, and this was effected by introducing two extra reinforced concrete piles to support the main beams at the north abutment, and two extra ones, united by a vertical reinforced concrete slab, at a position adjoining the south abutment; and a nearly horizontal reinforced concrete floor slab was put to connect this to the south abutment masonry. There is an opening of 6 ft. between the masonry of the south abutment and the adjoining piles to form a storm water overflow and give additional waterway in the case of floods.

The bridge is composed of three almost equal spans and the storm water overflow, the middle span being 32 ft. 4 in. centres, and the two adjoining spans being 31 ft. 10 in. respectively, and the storm water overflow 6 ft., giving a total length of 96 ft. between the abutment piles, or about 103 ft. 4 in. between the abutments. The width of the bridge, measured from the inside face of the parapets, is 10 ft., and a 12-in. stone kerb has been provided on either side and stone setts form the roadway.

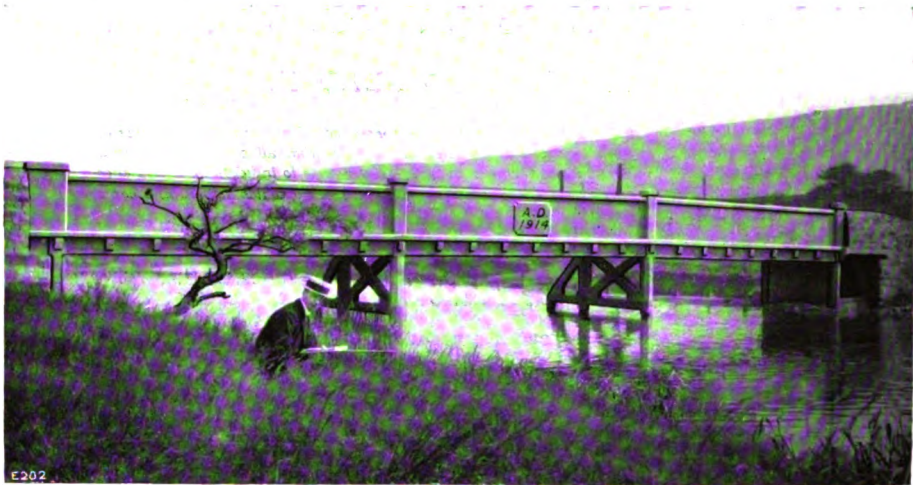
As this bridge is only for rural purposes, it was not deemed necessary to make it very wide or to provide for footpaths, the main object being to obtain a strong and economical structure. As the River Calder is subject to a considerable rise at certain times of the year, it was necessary to allow for as much water-way as possible, and the two intermediate piers, which are constructed on reinforced concrete piles, have been heavily braced as a protection against a strong current and any shock due to floating timber.

The piles were driven, some by means of a steam pile driver, but most by hand power, to an average set of $1\frac{1}{8}$ in. in the last ten blows of a 2-ton monkey falling from a height of 4 ft., and the average length of the piles from water level to point of pile is about 24 ft.

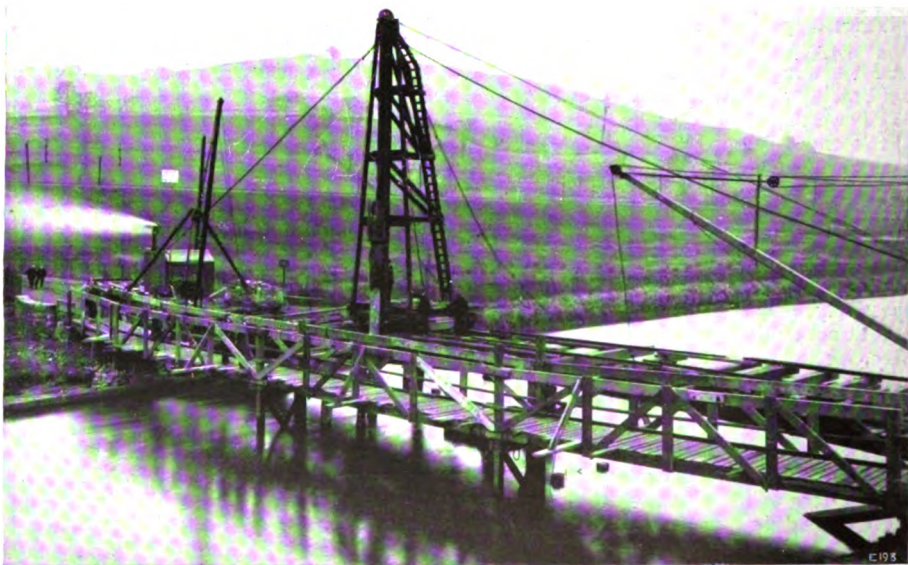
The bridge was tested by means of a heavy steam waggon and trailer, and practically no deflection whatever was noticed.

A timber bridge was erected temporarily to provide for foot and bridle traffic during the reconstruction of the bridge.

The contractors responsible for the execution of the work were the Yorkshire Coignet Contracting Co. (Messrs. Henry Atkinson and Sons, Ltd.), of Leeds.



Finished Bridge.



Bridge in course of construction.
REINFORCED CONCRETE BRIDGE OVER THE RIVER CALDER, YORKS.

NEW OFFICES FOR THE INDO-CHINA BANK, SINGAPORE.

THE new premises of the Banque de l'Indo-Chine, completed last year, are the first reinforced concrete buildings in Singapore. They are situated in Raffles Square, the centre of the town, where many old buildings have recently been demolished and replaced by new.

The total area of the site is about 4,000 sq. ft., and the ground is good, being chiefly hard clay and rock.

The design of the buildings is not well suited to reinforced concrete, the original idea having been to use a partial steel skeleton with steel and expanded metal floors. Reinforced concrete was ultimately employed, as its cost was substantially less than

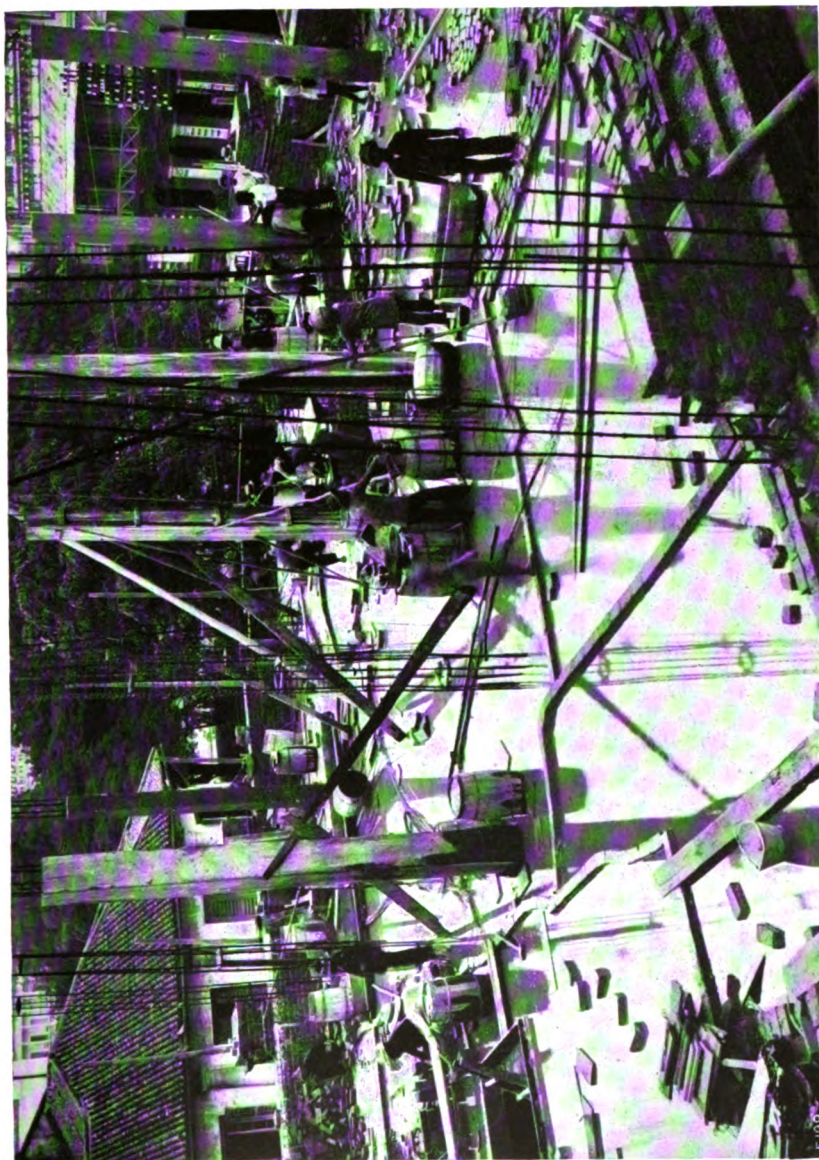


Fig. 1. Building in course of construction—first floor.
NEW OFFICES FOR THE INDO-CHINA BANK, SINGAPORE.

that of the lowest steel structure tender, but the general disposition of the building had to be maintained and the best possible use made of it.

The reinforced concrete structure comprises a general skeleton from the foundations, the pillars being imbedded in the brick piers and walls. The floors also are reinforced

concrete, the ground floor being specially reinforced in the centre, which forms the top of the strong room.

As soon as the first floor had been put into position, the bank took possession of the ground floor, and its business was carried on there during the erection of the remainder of the building. The first floor has a thickness of 5 in. only, the top inch

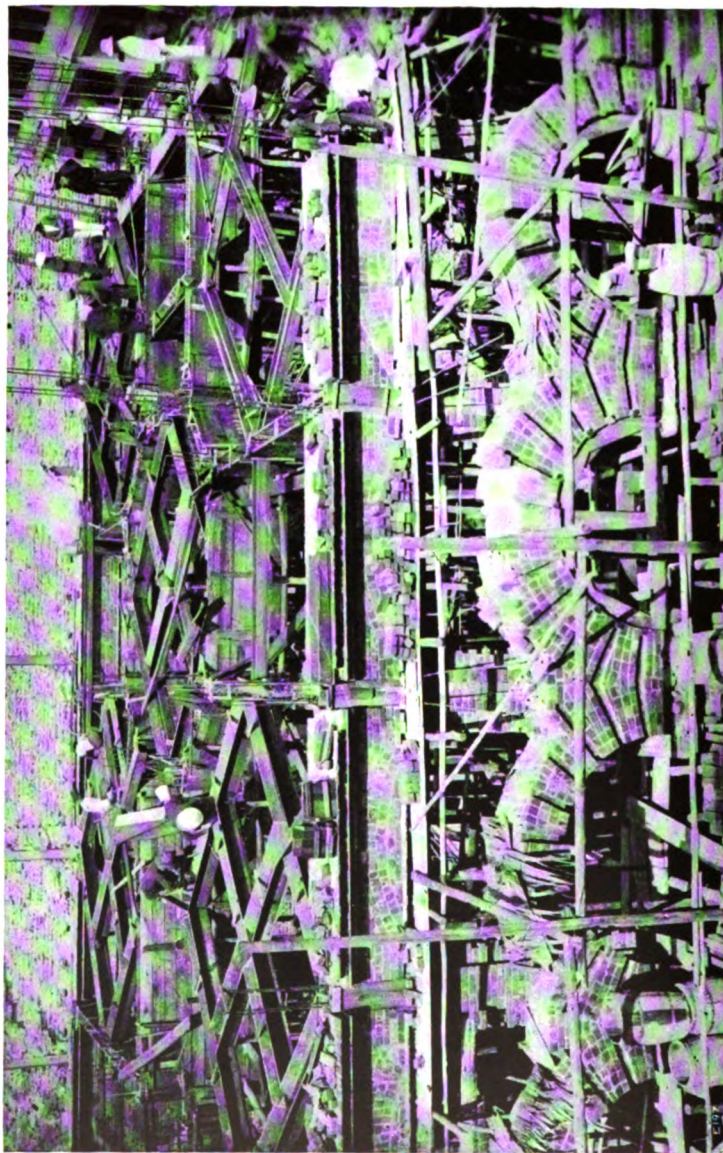


Fig. 2. Building in course of construction.
NEW OFFICES FOR THE INDO-CHINA BANK, SINGAPORE.

being 1-2 cement plaster, which was laid on the concrete directly the latter had been placed in position, and which thus forms a part of the floor. A dwarf wall was placed around the edge of this floor, which was then flooded with water and kept flooded for three months, when the roof of the building was completed. *Fig. 1* shows

the floor in this condition, and was taken when business was being transacted in the offices of the bank below in the ordinary way.

All the floors, staircases, lift stanchions, lintels, ties, etc., are of reinforced concrete, and so also are the roof trusses, which require no maintenance and are no more expensive than steel trusses.

The design of the ceilings, being somewhat elaborate, allowed reinforced concrete

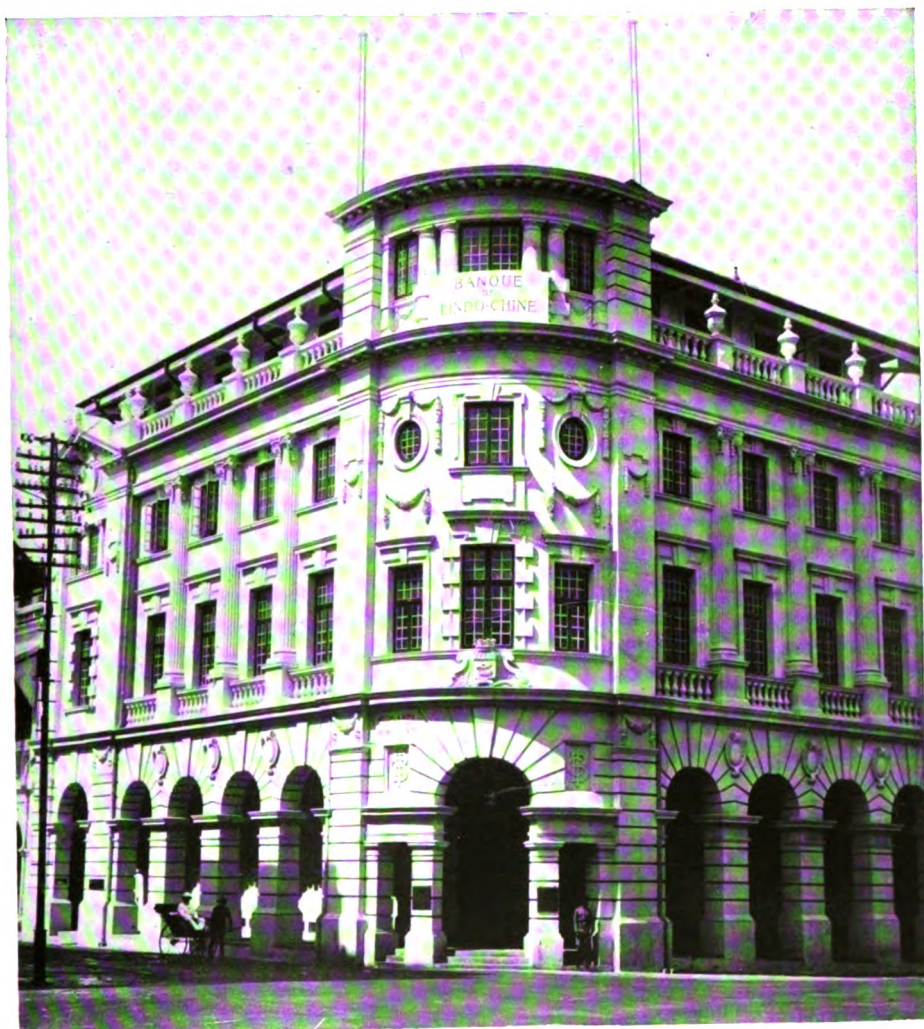


Fig. 3. Finished Building.
NEW OFFICES FOR THE INDO-CHINA BANK, SINGAPORE.

to show its adaptability to advantage. The main beams are 4 ft. deep and carry light brick filling, forming cornices. All ornamental work was cast in concrete moulds, concrete being used for external features and plaster-of-paris for internal. This is another new departure for Singapore, ornaments having previously all been made one by one and uniformity being therefore conspicuous by its absence.

The architect of the building was Mr. D. McLeod Craik, A.R.I.B.A., of Messrs. Swan and Maclaren, Singapore, and the contractors were Messrs. Brossard and Mopin, of Saigon, the whole structure being calculated and erected by their Singapore agent, Mr. P. H. Barriere.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

Construction of Masonry Dams. By **Chester W. Smith.**

London: The Hill Publishing Co., Ltd., 6 & 8 Boulevard Street, London, E.C. 279 pp. + xi. Price 12/6 net.

Contents.—Exploring the Site—Temporary Works for Stream Diversion—Preparing the Foundation—Masonry Construction—Quarrying—Face Work—Miscellaneous Features—Plant and Power—Installation Required and Power Consumption—Assembling Materials, Crushing and Mixing—Transportation of Materials—Probable Future Methods—Estimates of Cost—Partial List of Existing Dams, with Descriptions and Costs.

The author of this volume states that of all the structures designed and built by engineers, a masonry dam is probably the one about which there is the least amount of exact knowledge, and he gives his reasons for the statement and expresses them in a clear manner. The book is very practical in character, and the various aspects of the subject are dealt with in detail; thus the matter should be of value to engineers who are engaged in the design and construction of dams. The necessity of obtaining a thorough knowledge of the foundation is first considered, and notes are given which deal with borings; and this is followed by examples and types of diversion works and the preparation of the foundation. After describing the general construction of dams, the author gives two chapters on miscellaneous features, and these are probably the most interesting, if not also the most instructive, in the treatise. Such matters as uplift pressure, expansion joints, ice thrust, and initial stress are included under this heading, and actual examples are quoted when necessary to emphasise any particular point.

Many well-known dams are described in the last chapter, the descriptions being accompanied by drawings and photographs, together with details of the cost. The book is well written and fully illustrated, and we consider that the author has produced a comprehensive treatise which does justice to the subject, and we can thoroughly recommend it to our readers.

Plain and Reinforced Concrete Arches. By **J. Melan.** Translated by **D. B. Steinman, C.E., Ph.D.**

London: Chapman and Hall, Ltd., 11 Henrietta Street, W.C. 161 pp. demy 8vo. Price 8/6.

The book under review is a translation of the article by Professor Melan published in F. von Emperger's *Handbuch für Eisenbetonbau*, and contains the treatment of many problems that we have not seen dealt with in any other books on the subject.

The first five chapters deal with the conditions of stability of various arches for vertical loads derived from the elastic theory, and do not differ greatly from the usual mathematical treatment. The next chapter gives a brief treatment upon non-vertical loads.

Chapter VII. is upon approximate methods for the hingeless arch based upon the useful rule that the total area between the centre line of the arch and the line of pressure must be practically zero; from this he obtains a very useful result for the maximum bending moments for a uniform load over half the span. The next three chapters deal respectively with two-hinged and one-hinged arches, arches with elastic abutments, and the effect of continuity of arches.

Chapters XI. and XII. contain investigations of the determination of the best curve for and thickness for an arch, and one based upon a differential equation for the elastic line. A similar treatment has been given by Prof. Lilly, of Dublin, and deserves further recognition. The final chapter deals with the proportioning of reinforced concrete arches, and contains two worked examples and an interesting chart for facilitating the design of doubly-reinforced beams.

The book will undoubtedly be of considerable assistance to engineers who are already familiar with the formulæ resulting from the elastic theory of arches, and have sufficient facility in mathematical manipulation to follow the development of the formulæ for the special cases under consideration.

A large number of engineers, however, will in our opinion find the book very difficult to follow on account of the com-

parative lack of explanation of the mathematical formulæ; while it is true that some numerical examples are given, the explanation of such examples is rather meagre, and as a rule mere reference is given by way of number to the mathematical formulæ.

While recognising fully the difficulty of the task, we think there is still great need of a book which develops simpler formulæ that will express the results of the standard cases of loading with sufficient accuracy, making allowance for the necessary errors involved in applying to

reinforced concrete arches conditions that can be only approximate in practice.

In the case of the steel arch, the statical indetermination of the forces necessitates assumption as to the fixity of the ends and the deformation under stress, small departures from which have a considerable effect upon the results obtained; in the reinforced concrete arch we have, in addition to these difficulties, the inaccuracy due to the variation in the properties of the material and its effect upon the distribution of stresses in a doubly-reinforced beam.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The L.C.C. Regulations with respect to the Construction of Buildings wholly or partly of Reinforced Concrete.—At a meeting of the L.C.C. held last month the question of the regulations regarding reinforced concrete was under further discussion. The regulations as originally drafted have undergone considerable revision, and the regulations as revised were published in the Council's agenda of July 6th.

We give below a copy of the report of the Building Acts Committee as follows:—

On November 21st, 1911 (pp. 1170-1 and 1175-88), the Council, under the authority conferred on it by Section 23 of the London County Council (General Powers) Act, 1909, made regulations with respect to the construction of buildings wholly or partly of reinforced concrete. The regulations were prepared after consultation with the Institution of Civil Engineers, the Royal Institute of British Architects, the Surveyors' Institution, the Concrete Institute, and other associations and committees interested in the matter. The regulations, having been made by the Council, were submitted to the Local Government Board for allowance. After prolonged negotiations with the Board a revised draft of the regulations was prepared, and the Council on June 24th, 1913 (p. 1499), made regulations in accordance with this draft. The amended regulations were submitted to the Board for allowance, and in accordance with the provisions of the Act the four institutions before mentioned were informed of the Council's intention to apply for the allowance of the Board. Further negotiations have taken place with the Board and the institutions, and numerous suggestions for modifications in the regulations have been received. We think that a large number of these suggestions might be adopted, and we have accordingly prepared and now submit a revised set of regulations. The Local Government Board has stated that in its opinion the regulations as now amended might properly be allowed, and the Royal Institute of British Architects has placed on record its opinion of the great value of the regulations to architects not only in London, but throughout the whole of the Empire.

The regulations as now amended are less onerous than those made on June 24th, 1913, and more in line with the regulations made on November 21st, 1911. If, as the result of experience gained during the next few years, it should be found that alterations are desirable, there should be no difficulty in modifying the regulations.

In addition to the notice required by Section 23 (4) of the Act to be given to the above-mentioned institutions, the Local Government Board desires that, as in the case of by-laws to which Section 184 of the Public Health Act, 1875, applies, notice should be advertised in one or more London newspapers, preferably daily papers, and in one of the technical papers circulating amongst persons professionally or commercially interested or affected. The Board also desires that a copy of the regulations may be deposited for inspection at the offices of the Council for a clear calendar month after the publication of the advertisements and before the submission of the application to the Board for the allowance of the regulations.

If the Council approves the regulations, it will be necessary for them to be confirmed at a subsequent meeting. We recommend:—

(a) That the resolution (No. 1b) of June 24th, 1913 (p. 1499), making regulations with respect to the construction of buildings wholly or partly of reinforced concrete and with respect to the use and composition of reinforced concrete in such construction, and the further resolution (No. 5) of July 1st, 1913 (p. 3), confirming the same, be rescinded.

(b) That, pursuant to the provisions of Section 23 of the London County Council

(General Powers) Act, 1909, regulations with respect to the construction of buildings wholly or partly of reinforced concrete and with respect to the use and composition of reinforced concrete in such construction, be made as follows:—

(The regulations referred to will be published next month.)

(c) That notice be given to the Royal Institute of British Architects, the Surveyors' Institution, the Institution of Civil Engineers, and the Concrete Institute of the Council's intention to apply to the Local Government Board for allowance of the regulations specified in the foregoing resolution (b), and that notice be advertised in daily newspapers and technical papers circulating in London.

(d) That, on the confirmation of the regulations specified in the foregoing resolution (b) and after the expiration of one calendar month from the date of the appearance of the advertisements specified in the foregoing resolution (c), the necessary steps be taken to obtain the allowance of the regulations by the Local Government Board, and that the Board be asked to fix a date on which the regulations shall come into force.

A discussion took place on the report. Mr. Bernard Holland (Chairman of the Housing Committee) said that the report of the Committee ought to have been brought up in plain and clear language, so as to enable any member to understand the effect the regulations would have on the Council's dwellings for the working classes. He also asked whether the regulations would tend to increase the cost of buildings.

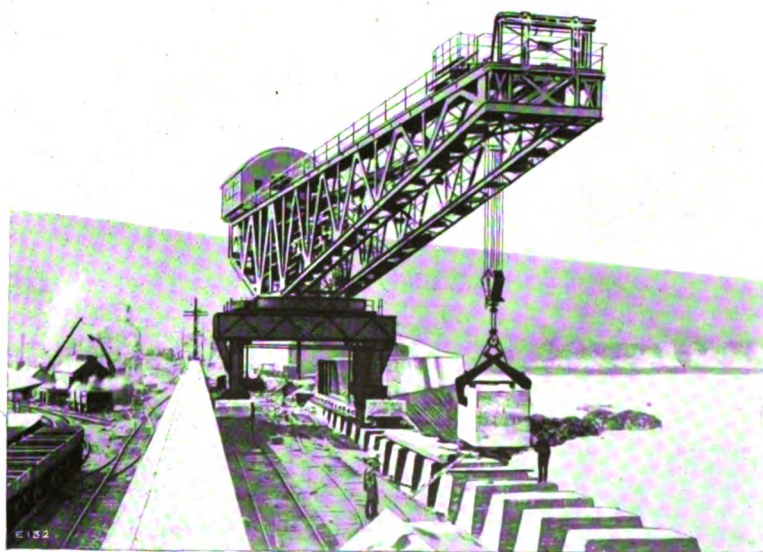
Major Levita enquired whether Government buildings in which reinforced concrete was used would be subject to the regulations. He thought the Government would not accept the regulations.

Mr. A. T. Taylor said the regulations only applied to large six, seven, or eight-storey buildings.

Mr. Dove did not think the regulations would affect workmen's dwellings at all. He thought the Council might let the report go through for what it was worth.

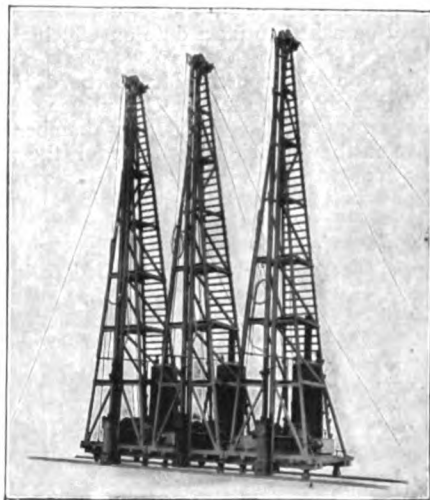
The report of the Committee and the regulations were adopted.

Concrete Blocks at Fishguard.—The accompanying illustration shows a concrete



block being laid in position by the "Titan" crane in order to reinforce the breakwater in connection with the Fishguard Harbour Station.

The BRITISH STEEL PILING CO



The above is a photograph of three complete Pile Driving Plants, as supplied by us to the Belgian Government lately. They were equipped with our 2-ton Single-acting Steam Hammers, 5-ton Double-barrelled Winches, and the pile driving frames were 55 ft. each.

When wanting reliable Pile Driving Plant for any purpose, send us your enquiries. Everything available for sale or hire, and immediate deliveries given from stock.

Weights of Piling on sale or hire :—

"SIMPLEX"	- -	22 to 27 lbs. per sq. ft.
"UNIVERSAL JOIST"		from 43 lbs. "

DOCK HOUSE, Billiter Street, LONDON, E.C.

The National Road Exhibition.—In connection with the National Road Conference, which was held at the end of June, there was an exhibition of road materials. Quite a number of firms exhibited, and we would specially refer to the exhibits of the following:—

British Reinforced Concrete Engineering Co., Ltd., who showed their road fabric, consisting of a drawn-steel wire mesh, which is electrically welded wherever the wires cross.

Ransome VerMehrs Machinery Co., Ltd., Brunswick House, Westminster, S.W., showed one of their well-known concrete mixers specially designed for comparatively small jobs. They also exhibited several lengths of their steel piling.

Sharp, Jones and Co., Parkstone, Dorset, showed their patent rock concrete tubes for sewers and culverts and their Aqualite manholes.

Stothert and Pitt, Limited, 38, Victoria Street, S.W., also exhibited a mixer known as the "Smith" mixer, capable of mixing either tar macadam or concrete.

Trussed Concrete Steel Co., Ltd., Caxton House, Westminster.—The principal exhibit of this firm was their "Hy-rib" steel sheeting and "Atlas" reinforcement for concrete road beds.

United States Steel Product Co., 36-38, New Broad Street, exhibited their triangle mesh concrete reinforcement.

George F. West Co., Caxton House, Westminster, exhibited their lock woven mesh for concrete reinforcement, as also samples of woven wire fencing.

All the exhibits have been described in detail from time to time in these columns.

The Exhibition was opened by the President of the Local Government Board, the Rt. Hon. Walter Long, M.P.

PROPOSED NEW WORKS.

Rochester.—Tenders have been invited by the Corporation for enclosing a railway arch with reinforced concrete to serve as a fire-engine station.

Sunderland.—By the Weardale and Consett Water Co.'s Bill, which has been passed by the House of Lords, it is proposed to construct a reservoir at an estimated cost of £400,000.

Tynemouth.—As the result of an agreement arrived at before a Parliamentary Committee, on the Water Bill of the Ashington Urban District Council, the Tynemouth Corporation have undertaken to promote a Bill for authority to construct a reservoir of a capacity of 2,000,000 gallons.

Hull.—Tenders have been called for during the month by the Corporation for reinforced concrete work in connection with the extension of the boiler plant at the electricity works.

Pirbright.—The Guildford Rural District Council have invited tenders for a bridge, with concrete abutments, over the canal at Pirbright.

Edinburgh.—In connection with the erection of their new generating station at Portobello, the Edinburgh Corporation have this month called for tenders for concrete foundation work, steel reinforcement, concrete piles, etc.

Ilminster.—The Somerset County Council are rebuilding Hem Bridge with a reinforced concrete arch, 13 ft. 6 in. in span, and have called for tenders for the work.

Durham.—Amongst the tenders called for during the month are three by the Durham County Council for laying reinforced concrete foundations on three sections of the Durham-Gateshead main road.

Rochdale.—The Water Committee propose constructing a reinforced concrete bridge at Cown.

Hastings.—Plans have been allowed by the Corporation for reinforced concrete foundations for the new block of the East Sussex Hospital. Mr. B. Green is the consulting engineer.

Merthyr Tydfil.—In connection with the waterworks extension of the Corporation tenders have been invited for a concrete balancing tank. Sir Alexander Binnie, Son, and Deacon are the engineers.

TENDERS ACCEPTED.

Colchester.—The Essex County Council have accepted the tender of Messrs. Myall Bros., Southend, of £882, for the construction of a reinforced concrete tank at Severells Asylum.

New Canadian Dock.—It is reported by the Imperial Trade Correspondent at Toronto that a new dock built by the Canadian Government, at a cost of £13,360, has been opened at Windsor, Ontario. The dock is of concrete throughout, and is 665 ft. long.

Openings in Shanghai.—H.M. Consul at Shanghai reports that it would be an advantage if some United Kingdom manufacturers would place stocks of bars, etc., in the hands of reliable representatives in Shanghai, and especially mild steel bars for reinforced concrete work, for which there is a constant and growing demand.

ERRATUM.

Recent Views: Stability of Quay Walls.—In our issue last month, page 363, we regret that in reporting the remarks made by Mr. Chas. F. Marsh, M.Inst.C.E., the latter part of his speech was inadequately reported. The report read: "Personally he should be perfectly satisfied with a factor of safety of 1.1."

His statement should have read as follows: ". . . If it were possible that all conditions, both external and internal, could be known, then a factor of safety of 1.1 would be sufficient."

TRADE NOTICES.

"Lapidolith."—"Lapidolith" is the name given to a colourless liquid compound which the makers claim wear-proofs and dust-proofs concrete floors permanently. Full particulars of this preparation can be obtained from the sole agents, Messrs. W. Macdonald and Co., Ltd., 23, Strutt Street, Manchester.

Waverley Book Co.—We understand that the Waverley Book Co. are giving away a sectional model of a building in reinforced concrete with every copy bought of their work on that important subject. "Reinforced Concrete," we are informed, is complete in one useful volume, and every section is written by a specialist, thus rendering the work one of the most complete and up-to-date publications on construction in concrete-steel on the market. It is supplied on terms that may be advantageous to those who are suffering temporarily from the effects of the war. We believe that the special offer of the sectional model is for a limited period only, so that any of our readers desiring further information should send a postcard, mentioning CONCRETE AND CONSTRUCTIONAL ENGINEERING, to the Waverley Book Co., Ltd., 7, 8, and 9, Old Bailey, London, E.C., asking for their useful free descriptive pamphlet.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



FRONT ELEVATION OF EXTENSION, WIGAN BATHS, MILLGATE.
(See page 473.)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X. No. 9.

LONDON, SEPTEMBER, 1915.

EDITORIAL NOTES.

THE NEED FOR RESEARCH WORK.

ELSEWHERE in this issue we give a short review of the Report for the year 1914-15 issued by the National Physical Laboratory, and a general idea of the work accomplished by the Engineering Department can be gathered from this review.

The question of research work is, however, of such primary importance that we take this opportunity of putting before our readers a few observations in the hope of stimulating more interest in the subject and thus possibly assisting in the future development of scientific investigation. The work actually done by the National Physical Laboratory is to be highly commended, and it is gratifying to find that the tests made in connection with the strength of materials alone for the past year number 241 as compared with 167 for the previous year, while the work proposed for the following year promises to be very useful, more especially to engineers, as the testing of reinforced concrete is included in the programme. The work, however, has been hampered by lack of funds, and it is a great pity to find that so little practical assistance is given by the State. The Laboratory fills a great need in this country, and yet it is not properly supported, with the consequence that it becomes necessary to send a letter to the Prime Minister asking him to receive a deputation to lay before him reasons for relieving the Royal Society from the financial responsibility involved in their control of the Laboratory and for placing the Committee in a position to carry out work urgently required by granting increased support from public funds. A Memorandum was drawn up, and this was sent, with a letter, to Mr. Asquith in July, 1914. The receipt of these were acknowledged, but the outbreak of war prevented further action. The financial position was so adversely affected, however, during the early months of the war that the Committee felt compelled to approach the Treasury for some assurance of support. It must be remembered that the Laboratory has been and is doing a great deal of useful work for the Admiralty and War Office during this critical time, and this fact alone is sufficient to justify generous support; while, apart from this consideration, we have before us the absolute necessity of making all possible progress in every branch of scientific research in order that we may successfully compete with other nations in all the world's industries, and there is no doubt that we have slumbered too long in this respect.

The Germans have probably realised more fully than any other nation the necessity of assisting their industries by scientific experiments, and they rightly consider that important improvements in any industrial process cannot be expected before exhaustive scientific research has been undertaken. The

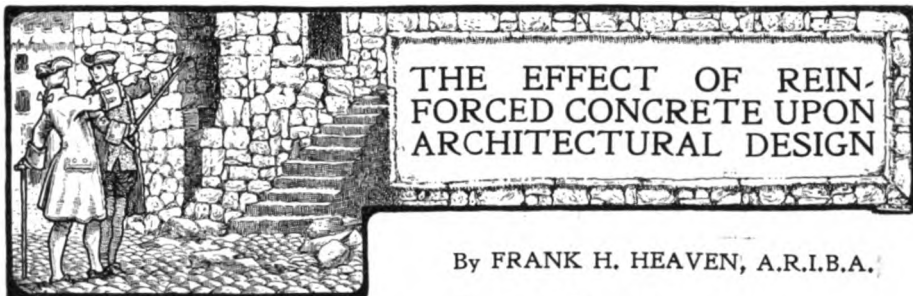
Americans have also advanced in this respect, and the Bureau of Standards at Washington, U.S.A., receives an annual grant of £100,000 from the Government, and again in France they are advanced far beyond Great Britain as regards research work. There are plenty of clever men in this country, but there appears to be a great deal of conservatism in Great Britain, and we always take a very long time before a real start is made in any step of advancement, the tendency to follow rather than lead being one of our greatest weaknesses.

Numerous questions are waiting to be investigated in relation to the building industry alone, and the need of research work was never more apparent than at the present time, when the application of scientific methods to building construction is being followed by engineers generally. The research work should, as far as possible, be conducted with the view of producing results of practical use in everyday work, and furthermore the investigations should be exhaustive and carried on to a definite end. A great number of experiments have been conducted in what may be termed a half-hearted manner, and the results are published in an incomplete form, so that very little benefit is conferred on the community. Again it would be advisable to devote our energies to those problems which have not been fully dealt with in other countries previously, and thus an endeavour made to produce new information of a useful character.

Research work is extremely interesting, and the investigator is usually well recompensed for his labours without regard to the actual money received; but it is unfortunately impossible to conduct experiments without going to some considerable expense, and it is only when funds are not limited that the best work can be done. The results of scientific investigation are of benefit to the whole of the nation, as they tend towards efficient and economical industry, and for this reason the National Physical Laboratory should be aided by the State and encouragement given in every way. Research work in relation to the scientific application of materials in building work has been greatly neglected in this country in the past, despite its great importance, and we hope that in the near future steps will be taken to deal seriously with the question by a real organisation of the powers and talent possessed by the various institutes connected with the building industry.

We certainly consider it the duty of the Treasury to assist the National Physical Laboratory liberally, and no question of war economies should be allowed to affect the work and the developments of this splendid institution. In fact, any parsimony on the part of the Government towards any of the scientific bodies or testing stations would be fatal. Where they meet with difficulties owing to the war they should be temporarily assisted with substantial grants, not only to carry on their useful work, but to prepare for the many problems that will arise in technical matters when the war nears its conclusion.

When peace has been declared we must be prepared for another form of warfare—namely, that of economics and industrial expansion, and we shall be hopelessly beaten if we allow our technical institutions to be hampered for lack of funds in the meantime.



The question of the effect of reinforced concrete upon architectural design is one of great importance and we trust, therefore, that the following article may prove of interest and use.—ED.

THE introduction and general use of any new material into the construction of buildings must, *ipso facto*, have its influence upon the architectural design of those buildings.

Reinforced concrete has now been in almost universal use for the past ten years or so, and its influence upon architectural design can be analysed by any person possessing an observant and retentive mind. It can be traced in the design of any building of size or importance by comparing first, its plan in the spacing and size of the points of support of superincumbent loads; and second, its elevation in the setting out of horizontal and vertical proportions, fenestration, projections ornamental and constructional, and general outline.

Let us look at the plan and note the effect of its use there.

Previous to what we will call the concrete age, there came the steel age during which the use of steel in beams and stanchions left its mark in the greater spacing of the points of support in buildings in which the free use of the floor area was a consideration, and the same difference can be traced, but in a greater degree, to the use of reinforced concrete. Before the steel and concrete ages the transmission of loads over voids was effected by wood beams (plain and trussed) and arches of brick or stone, but their use limited the distances between points of vertical support to very small spans indeed, owing to the low tensile resistance of the wood beam and the unsuitability of the arch for larger spans, except in the latter case, when due regard was given to the proportion of rise to span, which was not always possible; and also to the great outward thrust of flatter arches and the necessity for larger and stronger abutments to resist those thrusts than would be actually necessary if the load could have been transmitted vertically to the ground. Those vertical supports were therefore of necessity comparatively close together, of very large superficial area, and took up a large proportion of the available floor space.

This type of construction is well illustrated in the south wall of the nave of Ewenny Priory Church, Bridgend, Glam., which was commenced in the year 1100, where the piers are 3 ft. in diameter and the intercolumniation 9 ft. 6 in.

The support of loads carried by the external casing of a building necessitated the use of a continuous thick wall, or walls of medium thickness with piers or buttresses at the points where superincumbent loads had to be transmitted to the ground, and the use of piers at similar points of loading in the interior of a building. These buttresses or piers were consequently of large superficial

area owing to the limits of resistance to crushing of the material at hand—*i.e.*, stone (30 tons per sq. ft.) or brick (12 tons per sq. ft.).

Steel Construction.—With the advent of steel as a constructional material the areas of these points of support were reduced owing to the greater resistance of the steel to crushing (30 tons per sq. in.), and were capable of being placed at greater distances apart owing to the further use of steel in girders to carry the loads over larger spans; steel having an equal resistance to bending as to crushing. Hence the great resistance of steel to crushing reduced the area of the vertical support and the equal resistance of steel to bending had a further influence in widening the spans of openings in the walls externally, and the distance between points of support internally.

The objections to the use of steel in architectural design is that it is almost impossible to make it harmonise, in its plain constructional form, to æsthetic treatment; therefore it has to be cased up in brick, stone, terra-cotta, wood, or other material. But for the fact that this covering up of the steel excludes it from atmospheric action, to which action steel is very susceptible and yearly decreases in volume, by scaling, unless periodically painted or otherwise hermetically sealed from the air; its use as a constructional member when cased up would be a sham and without a place in true architecture.

An example of this type of construction is shown in the photograph of the new offices for the Law Union and Rock Insurance Co., Ltd. (Architect, Arthur C. Blomfield, Esq.), in which the spans between the stanchions are approximately 15 ft., 16 ft., and 17 ft. The author is indebted to Messrs. Redpath Brown and Co., Ltd., Constructional Steel Engineers, of London, etc., who supplied the steelwork in this instance, for the interesting photograph which shows the steelwork in course of construction and the method of casing the same in brickwork.

Reinforced Concrete.—The introduction of reinforced concrete, however, has brought to hand a material which can be used in its original form without any covering whatever, except perhaps for paint or colour necessary in the decorative treatment of the apartment in which it occurs. This combination of steel and concrete is sound construction, for here we have the concrete, which has a very high compressive resistance (600 lb. per sq. in.), offering resistance to the crushing effect of the load, and the steel, having a tensional resistance of 16,000 lb. per sq. in., embedded in the concrete, to take the tensional strain caused by the same loading. The embedding of the steel in the concrete effectively protects it from atmospheric action, and further, it can be so placed to advantage as to reduce to a minimum the quantity necessary effectively to resist the load to which the member may be subjected. Therefore the employment of these two materials, with opposite relative values, has resulted in a saving of metal as compared with the steel beam and stanchion, and also a saving of space, because the necessary covering for the steel is found in the construction of the member itself. The effect this has had upon architectural design is further to increase the spans of openings and distances between points of support by the introduction of reinforced concrete beams, and also to reduce the superficial area of vertical supports by the use of reinforced concrete pillars.

Floor Area.—Some idea of the uninterrupted floor area which may be

obtained by the use of reinforced concrete beams is to be seen in the photograph of the interior of a garage at Milton Hill (Messrs. Tubbs, Messer and Poulter, Architects), where the span of the main beams is 50 ft. and the spacing of the piers 20 ft.

Beams.—It is usual to design reinforced concrete beams so that there is a sufficient sectional area in the member to resist the crushing stress and to reinforce with steel, only for resistance to tensional strain; although steel reinforcement may be inserted into the compressive side where sufficient area of concrete is not provided. It is not, however, considered good design to reduce the size of a member so as to necessitate reinforcement on the compressive side, except where the exigencies of the design demand that the size of any beam must be kept within certain limits.

In vertical supports the steel is inserted into the position where it can



Fig. 1. Ewenny Priory Church, Bridgend, Glamorganshire.
THE EFFECT OF REINFORCED CONCRETE UPON ARCHITECTURAL DESIGN.

most effectively resist the compressive load placed upon it, strengthen the pillar against buckling or lateral flexure, and at the same time reduce the superficial area of the pillar to a minimum.

Reinforcement.—The convenient forms with which the steel reinforcement can be obtained, *i.e.*, in rods or bars, chiefly circular and of diameters varying in sixteenths of an inch from $\frac{1}{8}$ in. to 2 in., and the comparative ease with which the steel can be embedded and made to assume its proper position in the work, and also the economic cost at which the work can be carried out, has given reinforced concrete its place as a constructive material which must be considered by every architect in the design of buildings where it can be employed to advantage.

Elevational Design.—The general effect of reinforced concrete upon elevational design is to be seen in the tendency to breadth of treatment, due to

the vertical lines of support being less in number than previously, as pointed out above, and also to the widening of spans of openings and an almost entire elimination of the arch as a constructive feature. The heads of openings are now almost always flat by reason of the use of reinforced concrete lintels. Previously, where flat-headed openings had been desired, they were of limited span, owing to the very small resistance of wood or stone to bending or tensional strain. Now they can be made as wide as necessary without in any way impairing the strength of the building.

Fenestration.—Fenestration has also been modified by the same influence. Larger glass areas can be obtained with a minimum amount of walling; in

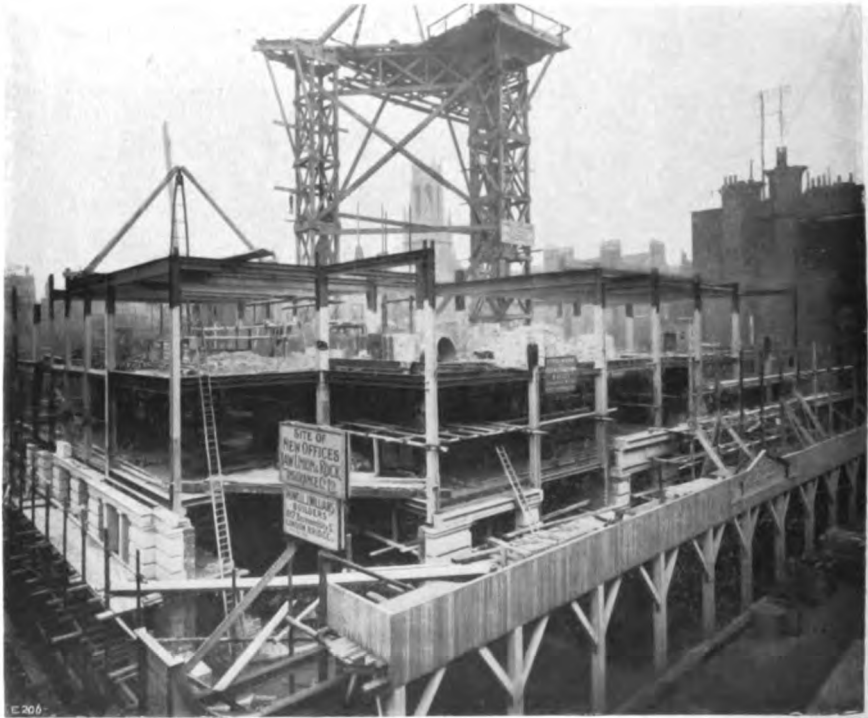


Fig. 2. Steelwork in Course of Construction.—New Offices for the Law Union and Rock Insurance Co., Ltd.
THE EFFECT OF REINFORCED CONCRETE UPON ARCHITECTURAL DESIGN.

fact, the infilling between the piers and beams of a modern reinforced concrete building might be all glass, should the necessity arise for such a consideration. The improvement in this direction is mainly to be observed in factories, schools, and buildings where the essential feature is the provision of ample light; but even in purely architectural building the influence is to be traced in a greater use of the flat-headed opening as compared with arched openings. True, some vitality of design may be lost by its employment, but with careful treatment the flat-headed opening may be equally an aid to beauty as the arch, and will certainly be an indication of the progress of thought in material.

Typical Examples.—Photographs of two typical factory buildings erected

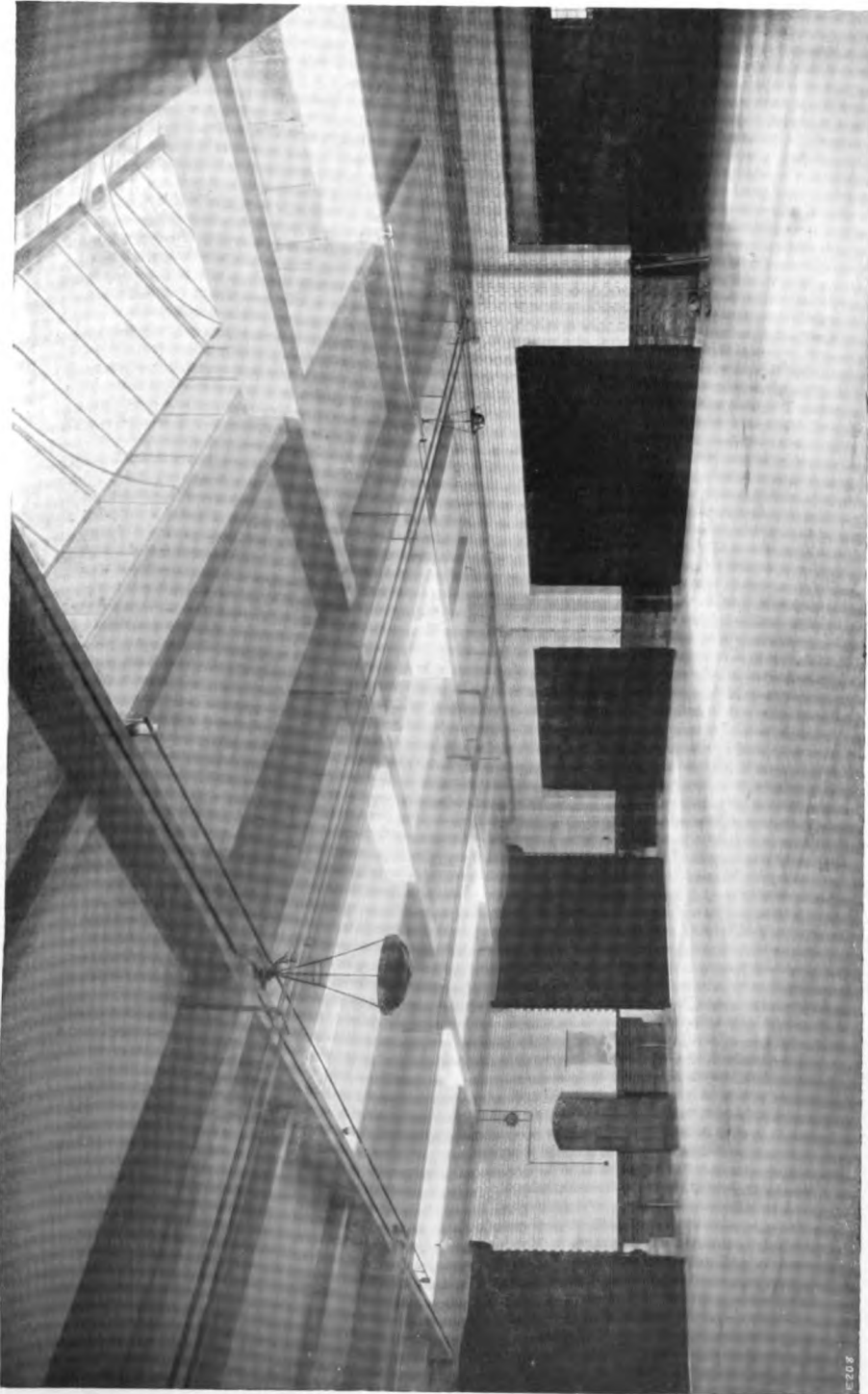


Fig. 3. Interior of a Garage at Milton Hill.
THE EFFECT OF REINFORCED CONCRETE UPON ARCHITECTURAL DESIGN.

in reinforced concrete are given which clearly illustrate the effect upon elevational design in breadth of treatment generally and the large glass areas obtained.

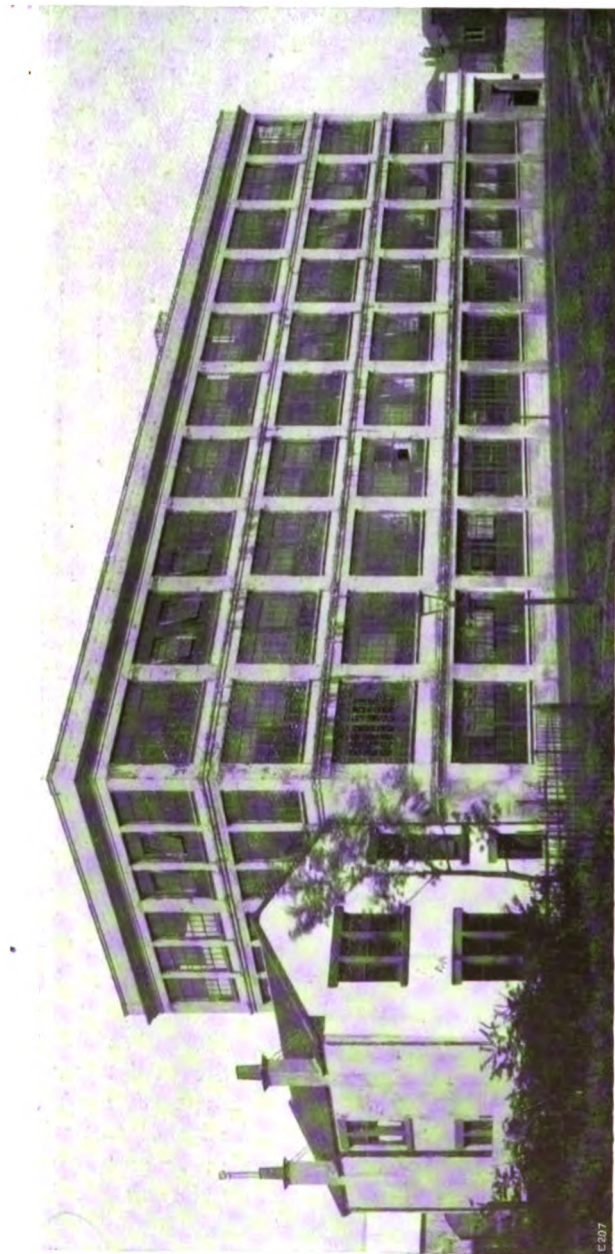


FIG. 4. Administration Block for Messrs. G. & J. Weir, Ltd., Cathcart.
THE EFFECT OF REINFORCED CONCRETE UPON ARCHITECTURAL DESIGN.

In that of the Administration Block for Messrs. G. and J. Weir, Ltd., Cathcart, Glasgow, the spacing of the columns on the front is 15 ft., centre to centre, and the main piers on the side 25 ft. A plan is given of this building showing the spacing of the interior supports and the very large uninterrupted floor space divided into two bays of about 150 ft. by 25 ft. by a centre row of columns at about 22 ft. 6 in. centres. In the New Motor Works for Messrs. Arrol-Johnston, Dumfries (T. C. Pullinger, Architect, and Messrs. Kerr and Watson, associate architects), the columns on all sides are spaced 18 ft. centre to centre. The author is indebted to Messrs. the Trussed Concrete Steel Co., Ltd., Caxton House, Westminster, S.W., for these photographs and details, from whose designs these buildings were entirely erected on the Kahn System of Reinforced

Concrete, and also for the photograph of the interior of the garage at Milton Hill, given herein.

Architectural Ornament.—It may be thought, perhaps, that reinforced concrete can have little or no influence upon the architectural ornament of a building, but its employment might greatly add to the grace of our building by producing a greater degree of light and shade upon elevational design by giving a large projection to strings, cornices, etc. The limits of stone or other granular material when in projection are well known, but ornamental reinforced concrete work could be employed to advantage where other materials fail. The Greek temples of old owe a great deal of their charm to the effects of light and shade cast by their projections. The sun, however, does not treat all lands alike. In southern Italy the proportioned cornice of the Orders produced the desired effect by reason of the high altitude of the sun, but the use of the same projection in England has an altogether different effect, due to the lower angle at which the light from the sun can produce shadows. Many of our monumental buildings are almost shadowless for that very reason. The architects of the Renaissance in Northern Italy recognised this fact and crowned their Palazzi with great overhanging cornices. The Americans, too, have seen the necessity for a deep shadow to give repose to their large scale buildings and obtain the same by the use of zinc or iron cornices of great projection, painted to imitate the material in which they



Fig. 5. Motor Works for Messrs. Arrol-Johnston, Dumfries.
THE EFFECT OF REINFORCED CONCRETE UPON ARCHITECTURAL DESIGN.

are working. The use of reinforced concrete in decorative cornices would be legitimate construction if we recognise the material and give the member its true value in relation to the whole design, and the deeper shadow cast by greater projections on our building would be appreciated.

Roof Construction.—Reinforced concrete has helped to perpetuate the flat roof and its deadly effect upon the general outline and mass of architectural building. The use of the stately pitched roof, so beloved for its reposeful and sub-conscious effect upon architecture, has been usurped by the use of the flat roof; not but that a flat roof is sound construction, and essential in some instances, but it is not a natural product of the climatic conditions of this

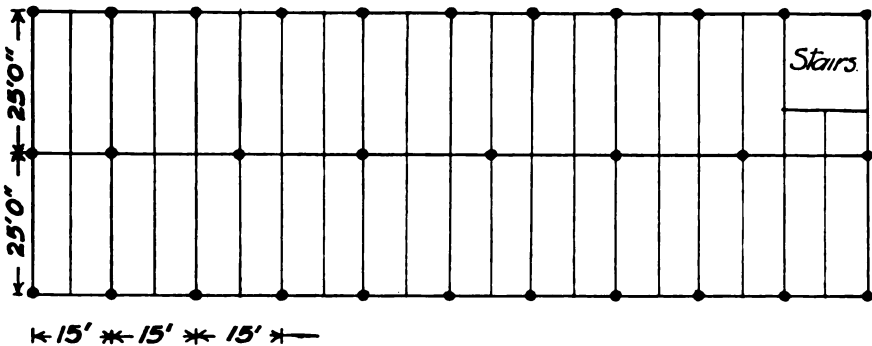


Fig. 6. Plan showing Lay-out of Columns in Messrs. Weir's Factory.
THE EFFECT OF REINFORCED CONCRETE UPON ARCHITECTURAL DESIGN.

country. The real reason for its use may be found in its fire-resisting qualities; but an equal safety may be obtained by constructing pitched roofs with a shell of reinforced concrete, in place of the usual timber framing, and covering it with the slates or tiles most suitable to the district for which the building is designed. To gain this security from fire the architects of the Middle Ages carried up the stone barrel vaults of their churches in solid masonry to the rake of the gable ends and bedded slates or stone slabs thereon as a protection from the weather.

Fire Resistance of Reinforced Concrete.—The almost perfect resistance offered by reinforced concrete to the effects of fire has had a great deal to do with its more general use in the design of buildings. It is essential that every architect should be well acquainted with all the properties of the many building materials at hand, so that he may be better able to make the best use of the same and render his work nearer to perfect architecture in that it is the best he can offer from the resources of his knowledge and the material at hand, for true architecture is to be found only in using the material at hand in a sound constructional manner; the result may be plain, but when the construction is intelligible there is no necessity to apply ornament to make it interesting.



In our last issue we referred editorially to the revisions made in the above Regulations, but owing to lack of space we had to hold over the actual publication of the Regulations themselves (as revised). We now give a part of them in this issue and the remainder will be published in two succeeding numbers.—ED.

PART I.

GENERAL.

Reinforced Concrete Defined.—1. The term “reinforced concrete” as occurring in these regulations shall mean concrete which is reinforced by metal so combined therewith that the metal will

- (a) be sufficient to take up all the tensile stresses;
- (b) assist in the resistance to shear;
- (c) assist in the resistance to compression where necessary.

Scope.—2. These regulations shall apply only to the construction of buildings of reinforced concrete in which the loads and stresses are transmitted through each storey to the foundations by a skeleton framework of reinforced concrete or partly by a skeleton framework of reinforced concrete and partly by a party wall or party walls.

Skeleton Framework.—3. The skeleton framework of reinforced concrete of a building together with the party wall or party walls (if any) upon which such framework bears shall be capable of safely and independently sustaining the whole dead load and the superimposed load bearing upon such framework and party wall or party walls, calculated in accordance with the data hereinafter contained.

Floors, Stairs, Roofs, etc.—4. All floors, stairs, landings, and other portions of a building carried by a reinforced concrete framework shall be constructed throughout of incombustible materials, and be carried upon supports of incombustible materials; nevertheless, all internal stairs and landings may be constructed of fire-resisting materials, and nothing in this regulation shall prevent the use of wood framing, boarding and battens in the construction of roofs.

Electrical Currents.—5. No part of the reinforcing metal in a building shall be used for conducting electrical currents.

Notices and Plans.—6. In the case of the erection of a new building of reinforced concrete or the making of any addition or alteration or the carrying out of other work under the provisions of these regulations, the like notice shall be served upon the district surveyor as is required to be served by Part XIII. of the London Building Act, 1894, and such notice shall be accompanied (a) in the case of a new building by plans and sections of sufficient detail to show the construction thereof, together with a copy of the calculations of the loads and stresses to be provided for and particulars of the materials to be used; and should such plans, sections, calculations or particulars as originally furnished be not in sufficient detail, the person depositing the same shall furnish the district surveyor with such further plans, sections, calculations or particulars as may be reasonably necessary for the aforesaid purpose, and (b) in the case of an alteration or addition or other work as aforesaid by such plans, sections, calculations and particulars as may be reasonably necessary.

PART II.

DATA TO BE USED FOR THE PURPOSES OF THESE REGULATIONS.

Floor and roof loads.

Dead Load Defined.—7. The dead load of a building shall consist of the actual weight of walls, floors, stairs, landings, roofs and all other permanent construction comprised in such building.

Superimposed Load Defined.—8. The superimposed load in respect of a building shall consist of all loads other than the dead load.

Loads.—9. In calculating the loads on foundations, pillars, piers, walls, framework, beams, and other constructions carrying loads in buildings, the superimposed load shall be estimated as equivalent to the dead load set forth in the regulations following, and numbered *ten to twenty-four*, both inclusive.

Floor Loads.—10. For a floor the estimate shall be made in accordance with the table immediately following :—

For a floor intended to be used for the purpose of	Equivalent dead load in lbs. per sq. ft.
Domestic buildings not hereinafter specified	70
Other similar purposes	
Asylum wards	84
Common lodging-house bedrooms	
Hospital wards	
Hotel bedrooms	
Workhouse wards	
Other similar purposes	
Counting houses	100
Offices	
Other similar purposes	
Art galleries	112
Chapels	
Churches	
Classrooms in school buildings, not being dwelling houses so used	
Lecture rooms	
Meeting rooms	
Music halls	
Public assembly	
Public concert rooms	
Public library reading rooms	
Retail shops	
Theatres	
Workshops	150
Other similar purposes	
Ball rooms	
Drill rooms	
Other similar purposes	224
Similar floors subject to vibration	
Book stores at libraries	
Museums	224
For every floor in a building of the warehouse class except such floors as are intended to be used for any of the purposes aforesaid—not less than	

Loads on Stairs and Landings.—11. For stairs and landings the superimposed load shall be estimated as equivalent to a distributed dead load of at least 120 lb. per sq. ft., but each step shall be capable of safely supporting a concentrated load at any point of not less than 300 lb.

Loads on Roofs.—12. For a roof the plane of which inclines upwards at a greater angle than 20 degs. with the horizontal, the superimposed load, which shall for this purpose be deemed to include wind pressure and weight of snow and ice, shall be estimated at 28 lb. per sq. ft. of sloping surface, normal to such sloping surface, on either side of such roof.

13. For all other roofs the superimposed load shall be estimated at 56 lb. per sq. ft. measured on a horizontal plane.

Greater Loads.—14. If the superimposed load on any floor, landing, stairs, or roof is to exceed that hereinbefore specified for such floor, landing, stairs, or roof, such greater load shall be provided for with an equivalent margin of safety.

Loads not Otherwise Specified.—15. In the case of any floor or roof intended to be used for a purpose for which a superimposed load is not specified in these regu-

lations, the superimposed load to be carried on such floor or roof shall be provided for with an equivalent margin of safety.

Rolling Loads.—16. In cases where a rolling load actuated by mechanical power is to be provided for such rolling load shall be taken as equivalent to a static load 50 per cent. in excess of the actual rolling load. The positive and negative bending moment at every cross-section due to every position of the rolling load shall be properly provided for.

Partitions.—17. Partitions and other structures superimposed on floors and roofs may be included in the superimposed load, provided the weight of the partition or other structure per sq. ft. of base does not exceed the permissible load per sq. ft. of floor or roof area. Partitions and other structures of greater weight shall be specially provided for.

Angle of Dispersion.—18. For calculating the resistance moment the angle of dispersion of a point load through hard filling and concrete shall not be taken at more than 45 degs. from the vertical.

Loads.—19. For the purpose of calculating the total load to be carried on foundations, pillars, and walls in buildings of more than two storeys in height, the superimposed loads for the roof and for the floor of the topmost storey shall be calculated in full, but for the floors of the lower storeys a reduction of the superimposed loads shall be allowed as follows:—

For the floor of the storey next below the topmost storey a reduction of 5 per cent. of the superimposed load on the floor of that storey, as calculated in pursuance of the foregoing regulations, shall be allowed; for the floor of the next storey 10 per cent. of the superimposed load on the floor of that storey similarly calculated; and so on by increments of 5 per cent. per storey till the reduction amounts to 50 per cent. It shall be taken at 50 per cent. per storey for all floors below.

20. No such reduction as aforesaid shall be allowed in the case of a building of the warehouse class.

Wind pressure.

Wind Pressure.—21. All buildings shall be so designed as to resist safely a horizontal pressure equivalent to a static pressure of not less than 20 lb. per sq. ft. of the whole projected surface normal to the direction of every wind.

(For pressure on panels of external walls see regulation 130.)

22. All structures or attachments whatsoever in connection with a building, including towers or other parts which extend above the roof, flat, or gutter adjoining thereto, shall be so designed as to resist safely a horizontal pressure equivalent to a static pressure of not less than 40 lb. per sq. ft. of the whole projected surface normal to the direction of every wind.

Working load.

Working Load.—23. The working load shall include the superimposed load, and the dead load inclusive of the weight of the beams, slabs, pillars, or other members, and any plastering, tiles, mosaic, granitic or other similar finishing materials.

Weight.

Weight.—24. For the purposes of calculation, the average weight of reinforced concrete, together with any finishing materials as aforementioned, shall be taken at not less than 144 lb. per cu. ft. measured over finished surfaces.

Ratio of span to depth of a beam.

Ratio of Span to Depth.—25. For the purpose of determining the ratio of span to depth of a beam, the effective depth of the beam shall be taken.

26. The effective depth shall be measured from the compressed edge of the constructional concrete to the common centre of gravity of the tensile reinforcement.

27. The ratio of the span of a beam to its effective depth shall not exceed the lesser of the two following ratios:—

$$20 \times \frac{\text{tensile stress in regulation 43}}{\text{actual maximum tensile stress}}$$

or

$$20 \times \frac{\text{compressive stress in regulation 42}}{\text{actual maximum compressive stress}}$$

28. The length of a cantilever shall not exceed five times the effective depth at the bearing.

Bending moments.

Effective Span.—29. For the purpose of ascertaining the bending moments on a beam or on a slab, the effective span and the whole load on the effective span shall be taken into account in the calculations.

30. In the case of non-continuous beams or slabs, the effective span shall be taken as the distance between the main vertical sides of the piers, pillars, or walls, plus the effective depth of the beam or slab at the supports, or the span between the centres of the necessary bearing surfaces, whichever may be the lesser.

31. In the case of continuous beams or slabs the effective span shall be taken as the clear span plus the effective depth of the beam or slab, or the length between the centres of the supports, whichever may be the lesser.

Fixed Ends Defined.—32. A beam or slab shall be deemed to have fixed ends when its ends are sufficiently secured to other parts of the construction having such rigidity as will maintain the neutral planes of the beam at the ends in their original positions and directions under all variations in the incidence and intensity of loading.

Maximum Moments.—33. The bending moments to be provided for at every cross-section of a beam or of a slab shall be the maximum positive and negative moments at such cross-section. Such maximum bending moments shall be calculated in a manner consistent with regulations 34, 35, 36, 37, and in accordance with regulations 38 and 39.

BEAMS WITH ENDS FREE OR FIXED.

Notation for bending moments.

34.

B = bending moment.

b = breadth.

l = length of effective span.

W = total weight or working load.

Load.	Supports.	Value of B to be provided for.
Concentrated at free end of cantilever	Cantilever	$B = -Wl$ at fixed end.
Uniformly distributed ..	Cantilever	$B = -\frac{Wl}{2}$ at fixed end.
Concentrated at centre of beam	Both ends freely supported	$B = \frac{Wl}{4}$ at centre.
Uniformly distributed ..	Both ends freely supported	$B = \frac{Wl}{8}$ at centre.
Concentrated at centre ..	Both ends fixed	$B = \frac{Wl}{8}$ at centre.
Concentrated at centre ..	Both ends fixed	$B = -\frac{Wl}{8}$ at ends.
Uniformly distributed ..	Both ends fixed	$B = -\frac{Wl}{12}$ at ends.
Uniformly distributed ..	Both ends fixed	B at centre shall not be taken at less than the numerical value of B at the ends.
Uniformly distributed ..	One end fixed One end freely supported	$B = -\frac{Wl}{8}$ at fixed end.
Uniformly distributed ..	One end fixed One end freely supported	$B = \frac{Wl}{14}$ at $\frac{3}{4}$ of span from the free end.

CONTINUOUS BEAMS.

Notation for bending moments.

Continuous Beams.—35. B = bending moment.

l = length of effective span measured between the centres of supports. (See regulation 31.)

W = total weight or working load. (Superimposed and dead loads.) (See regulation 23.)

W_d = total deadweight per span.

W_s = total superimposed weight or load per span (uniformly distributed).

The total bending moment (B) to be provided for at every cross-section shall be the algebraic sum of the bending moments due to the superimposed and dead loads respectively.

Maximum bending moments due to variations in the incidence of distributed loads over approximately equal spans.

Near middle of end span.	At support next to end support.	At the middle of interior spans.	At other interior supports.
$+\frac{W_s l}{10} + \frac{W_d l}{12}$	$-\frac{W_s l}{9} - \frac{W_d l}{10}$	$+\frac{W_s l}{12} + \frac{W_d l}{24}$	$-\frac{W_s l}{9} - \frac{W_d l}{12}$
or approximately $+\frac{Wl}{10}$	or approximately $-\frac{Wl}{10}$	or approximately $-\frac{Wl}{12}$	or approximately $-\frac{Wl}{10}$

36. Notwithstanding anything in regulation 35, beams may be designed for the exact positive and negative bending moments which will occur at every cross-section whether all the spans be loaded or alternate or any of the spans be unloaded.

Bending Moments on Slabs.—37 (a). Bending moments on slabs supported by, fixed at, or continuous over four edges, shall be calculated as for beams.

(b) When reinforced with mesh reinforcement or reinforced in two directions at right angles to each other, the load assumed to be carried in each direction may be apportioned as follows:—

Proportion of load assumed to be carried in the direction of the *breadth* of the slab—

$$= \frac{1}{1 + \left(\frac{b}{l}\right)^4} \times \text{total load on slab.}$$

Proportion of load assumed to be carried in the direction of the *length* of the slab—

$$= \frac{1}{1 + \left(\frac{l}{b}\right)^4} \times \text{total load on slab.}$$

Provided that when the length of a slab exceeds twice its breadth the whole load shall be assumed to be carried in the direction of the breadth of such slab.

Moments over Supports.—38. The maximum area of steel required to take the negative bending moment at any support shall be carried without reduction across the centre of that support. (Also see regulation 35.)

Variations of Loading.—39. To allow for variations in the incidence of the loading on contiguous spans of continuous beams or slabs, the bending moments to be provided for at every cross-section shall be the maximum positive and negative moments at such cross-section, whether all the spans be loaded or alternate or any of the spans be unloaded.

Conditions of Loading not Otherwise Specified.—40. For every condition of loading not specified in these regulations, the bending moment for beams or slabs shall be calculated so as to give an equivalent margin of safety.

Reinforcement at Points of Contraflexure.—41. Reinforcement shall be carried beyond the points of contraflexure under any condition of loading, by a length at least equal to half the effective depth of the beam.

Working stresses.

Stresses in Concrete. Also see regulations 145, 148, 156, 157, 158, 159, 162—42 (a). Except as further provided for in pillars, the permissible working stresses in concrete shall not exceed the following:—

Stresses in concrete.	Proportion by volume.											
	Cement.	Sand.	Coarse material.	Cement.	Sand.	Coarse material.	Cement.	Sand.	Coarse material.	Cement.	Sand.	Coarse material.
	1	2	4	1·2	2	4	1·5	2	4	2	2	4
	V = 6			V = 5			V = 4			V = 3		
	Lb. per sq. in.			Lb. per sq. in.			Lb. per sq. in.			Lb. per sq. in.		
* Direct compressive stress ..	600			650			700			750		
Extreme flexural compressive stress in beams	600			650			700			750		
Shearing stress	60			65			70			75		
Grip or adhesion between concrete and steel bars hooked at both ends	100			100			100			100		
Grip or adhesion between concrete and steel bars otherwise effectively anchored at the ends	60			60			60			60		
Tensile stress	nil.			nil.			nil.			nil.		

* See regulation 100.

(b) The values of the permissible *compressive* stress in pounds per square inch for intermediate proportions may be estimated from the following equation:—

$$c = 900 - 50 V$$

where *V* equals the *volume* of the sand plus that of the coarse material, per volume of cement, each measured separately and including the voids proper to each material.

If *V* is less than 3, the stresses in the last column shall not be exceeded.

(c) The values of the shearing stress shall not exceed one-tenth of the direct compressive stress for any given value of *V*.

Stress in Steel.—43. The permissible working stress in the steel shall not exceed the following:—

Stresses in mild steel complying with the British Standard Specification.	Lb. per sq. in.
Compressive stress	<i>m</i> times the stress in the concrete immediately surrounding the steel (the value of <i>m</i> being obtained in accordance with regulation 54). See also regulations 61 and 62.
Tensile stress	16,000

Eccentric Loading.—44. Provision shall be made for the stresses due to eccentric or non-axial loading.

Combined Stresses.—45. The term "combined stresses" when used in these regulations shall mean the resultant of all the stresses due to any cause or causes.

46. Neither the steel nor the concrete shall be subjected to combined stresses which exceed the permissible working stress.

Connections.—47. All connections in or between reinforced concrete members shall be designed and arranged so that the stresses which may come upon them shall be within the limits allowed by these regulations.

Hooks and Anchors.—48 (a). All tensile and shear reinforcement shall be hooked or otherwise effectively anchored at each end.

(b) Hooks at the ends of bars shall be of a  form, and shall have an inner diameter of at least four times the diameter of the bar; or when the hook fits over a main reinforcing bar the diameter of the hook may be equal to the diameter of such bar. The length of the straight part beyond the end of the curve shall be at least four times the diameter of the bent bar.

(c) When the bars are anchored at each end by means other than a hook complying with paragraph (b) of this regulation, the anchorage shall be bent up to a right angle from the centre line of the bar. The width across such anchorage shall be at least three times the normal diameter of the anchored bar. (Also see regulation 139.)

Grip length.

Grip.—49. The grip length or adhesion length of a bar embedded in concrete shall be measured along the bar from the beginning of the curve of the hook or the nearer end of the other form of anchorage.

50. The grip or adhesion length shall be sufficient to keep the stresses within the limits set out in regulation 42, except for web reinforcement complying with regulations 48 (b) and 67.

51. For the purpose of ascertaining the necessary grip or adhesion length in the case of a bar having mechanical bond the perimeter of the bar may be measured over the transverse projections provided—

(a) That the transverse projections are not further apart (centre to centre) than twice the normal diameter of the bar;

(b) That the height of such projections above the normal surface of the bar shall be at least 1-10th of the diameter of the bar.

Modular Ratios.

Modular Ratios.—52. The term "modular ratio" (m) means the ratio of the elastic modulus of steel to the elastic modulus of the concrete.

53. The elastic modulus for steel in tension or compression shall be taken at thirty million pounds per square inch. (See regulation 80 (c).)

See Regulations 83 and 114.—54. The modular ratio for steel and concrete shall be taken as follows:—

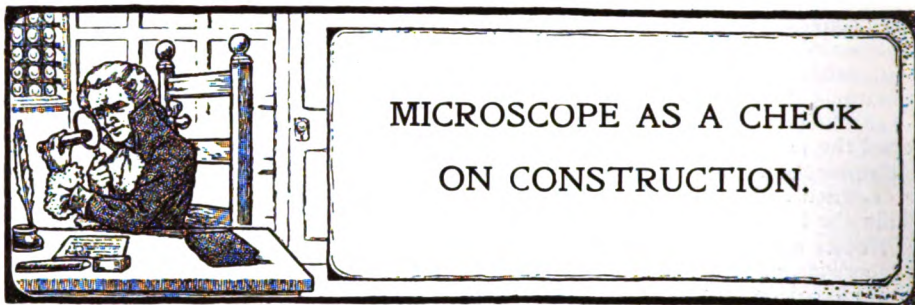
For beams $m.c. = 9,000$ or $m = \frac{9,000}{c}$

For pillars with the minimum of hooping or binding $m.c. = 9,000$ or $m = \frac{9,000}{c}$

For pillars with more than the minimum of hooping
or binding $m.i. = 9,000$ or $m = \frac{9,000}{i}$

Where c = permissible compressive working stress given in regulation 42 and
 i = increased stress given by regulation 115.

(To be continued.)



Demonstrating the Quantitative Mechanical Analysis of New or Old Concretes.

By **NATHAN C. JOHNSON**,
Consulting Engineer, New York City.

Fourth article of series.—In reprinting these articles we would state that the opinions expressed are those of the author, and the Journal as such does not necessarily associate itself with the conclusions arrived at. The articles are reprinted by the courtesy of the "Engineering Record," U.S.A., and the illustrations have been placed at our disposal by the author.—ED.

In any examination of concrete constructions, where the quality of the product may be in doubt, the detection and localisation of serious defects are matters of prime importance. In preceding papers certain imperfections as revealed by microscopic examination of typical samples of the mixture have been dealt with. In this article the detection of weak and faulty mixtures, resulting from either dishonesty or carelessness in field mixing, will be considered, with special reference to their bearing on the failure of completed structures.

CONTROL NEEDED.

The more the import of even a single failure is considered, the more desirable it seems that some means of controlling the product should be at hand during the making of concrete, instead of waiting until it is too late before serious defects at critical points are detected. Further, in case the work is done under contract and there should be suspicion that materials were being used in improper proportions, as, for instance, too great a quantity of aggregate for the quantity of cement, it is very desirable that some means other than a tally of empty cement sacks be available to decide the disputed point. Or, in case of the collapse of a concrete structure, it is highly advantageous in the determination of causes that the proportions and quality of the set concrete should be known, independently of the specifications that were supposed to govern its building, or of the honest beliefs of the constructor,

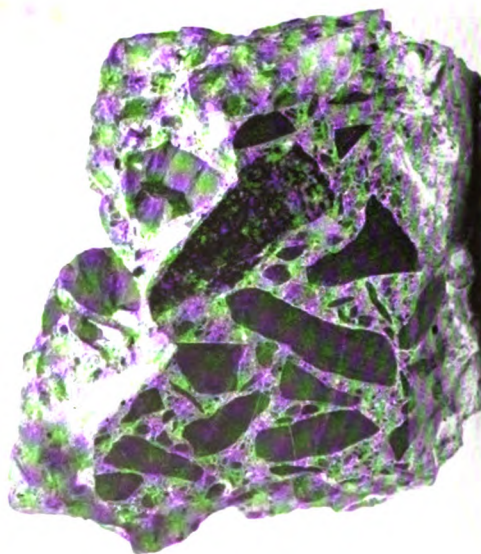


Fig. 17. Surfaced Specimen of Concrete.
One-fourth natural size.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

for specifications usually are interpreted generously, and all mankind is prone to inadvertent error. Many other uses suggest themselves for a reliable, simple method of analysis, which will be applicable either to old or new concretes, according to the needs of individual cases.

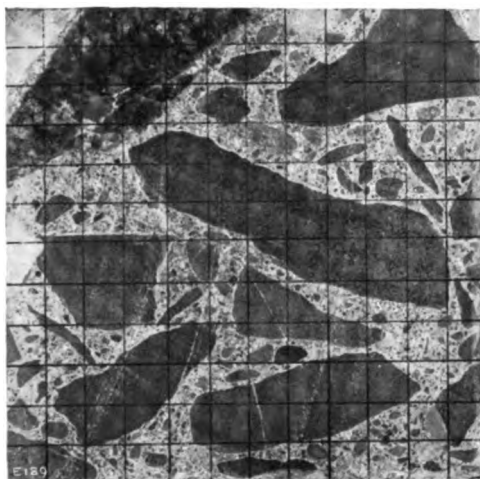


Fig. 18. Photograph of Specimen cross-ruled.
One-half natural size.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

that it was a random section. In the micro-photographs of mortars in Article 1 and in the articles following the same assumption was made, and it is fair to assume that this is correct, provided other sections from different parts of the same structure are examined without showing too great a variation. Experience seems to demonstrate that where proper care in mixing and placing has been exercised, there is but slight variation in sections from different portions of any given structure. This applies equally to gross and to micro-structures. On the other hand, where carelessness has prevailed sections are quite dissimilar, so that a sufficient number of samples must be sectioned to get a proper average. The obtaining of representative data is a first principle in any work.

Granting, however, that representative samples are obtained, and that the random surfaces are typical of the mass, it is then necessary to make the further assumption that areas are equivalent to volumes, both as regards the large aggregate, the small aggregate, and the cement, both hydrated and unhydrated.

C2

It requires but a little reflection to see that a variation of the same sort of microscopic examination that has previously been demonstrated will furnish not only the desired check on construction, both in materials and methods, but also the means for a quantitative analysis of practically any concrete of any age. In the latter application it is necessary at the outset to make certain assumptions, but if they are proved correct by the results of the actual analysis, they may be considered to be justified.

ASSUMPTIONS MADE.

In Article 1 of this series a surfaced section of a field concrete was shown, and the structure of this section was held to be representative of the structure of the entire mass, on the ground

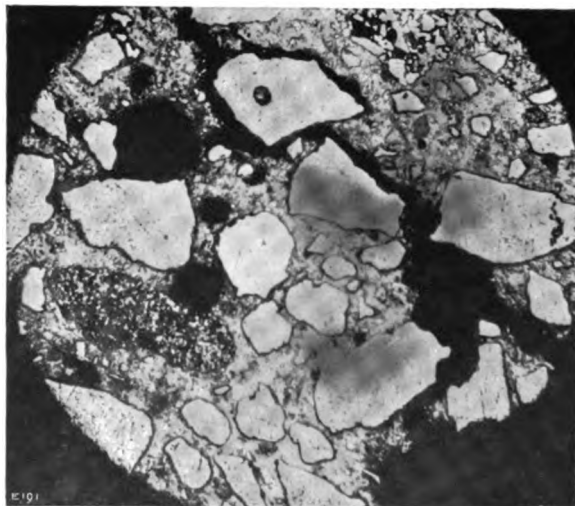


Fig. 19. Photo-Micrograph of Specimen FIG. 18. Enlarged to 20 diameters.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

It is evident that this must be true if the first assumption is correct—i.e., that the surfaces obtained are typical of the structure of the mass. And if this second assumption is true, it is possible to obtain in a gross section the ratio of volume of [large aggregate] to volume of [sand + (cement + water + air)] and in a small section, containing no large aggregate, it is possible to obtain, by using the microscope to give an enlarged image, the ratio of the volume of [sand] to volume of [cement + water + air]. This is the basis of this method of analysis.

ANALYSIS DEMONSTRATED.

In order to demonstrate the correctness of these assumptions by the working out of the analysis, it will be simplest to use a laboratory-mixed concrete for a trial.

Fig. 17 shows a sectioned lump of such a concrete about 8 in. on a side and weighing about 10 lb. This cut is about one-quarter natural size. The characteristic wide dispersion of aggregate, which has become familiar in the examination of other concretes, is evident at first glance.

Fig. 18 shows a portion of the same section, enlarged to half natural size. Even the smaller aggregates are plainly seen. It will be noted that these smaller grains, also, are out of contact with each other. Evidently any void determinations that were made on these aggregates would most certainly be wide of the mark, so far as the present inter-aggregate volumes are concerned.

In Fig. 19 a magnified portion of the same surface is shown photographed through the microscope. All of the large aggregate has been excluded, as even a portion of one of the larger pieces would completely cover the field. The "rocks" showing in this picture are sand grains, as has before been pointed out. Characteristically, even the smallest of these are widely dispersed, with large lumps, or groups of particles, which are unhydrated cement, lying between them just as in other concretes that have been previously examined. These features have doubtless long since become so familiar to readers of this series as to be instantly recognised.

DETERMINING AREAS.

Evidently, by means of magnifying with the microscope, even the smallest kind of sand grains may be rendered not only distinctly visible, but as of such size that their areas can be determined by appropriate means; and, going a step further, the ratio of their combined areas to the area of the total field can be had by simple addition. Further, when this addition is completed, subtraction of sand areas from the total area of the field (a convenient circle) will give the area taken up by [cement (hydrated or unhydrated) + water + air]. Or calling areas equal to volumes in each case, this arithmetic gives directly the ratio of [sand] volume to [cement + water + air] volume. Ratios determined in this manner for a number of fields sufficient to give a fair average, should give, as an average, the true ratio of the volume of [sand] to the volume of [cement + water + air] in any concrete.

To take the actual case: As the simplest way of determining areas, rule the surface of Fig. 18 into small squares. By counting the squares taken up by the stones and deducting from the total number of squares, the ratio of volume of [stone] to the volume of [sand + (cement + water + air)] should be obtained. This is found to be as 67 to 144, or the volume of the stone is 46.5 per cent. of the mass.

In determining sand volumes the procedure is slightly more complicated. In Fig. 20 is shown a composite photograph obtained by matching together a number of small photographs, taken at low power through the microscope and with a co-ordinate screen in the eye-piece to give the section lines. The actual surface area on the specimen covered by this photograph is 100 sq. mm., or a square 10 mm. on a side, and as this area is very small it would be necessary to secure a proper average to cover several such areas. For the present, however, an illustrative case only is

desired, so that this one area will suffice. It is evident also that in counting areas in such a field the percentage of error will be proportionately low as the degree of magnification is increased. For this reason, and also in order that the very smallest sand grains may be taken account of, a magnification of about 60 diameters, rather than the lower power here used, is usually desirable. In this manner, by counting the sand areas and deducting from the total area of the field, the ratio of volume of [sand] to volume of [cement + water + air] is obtained.

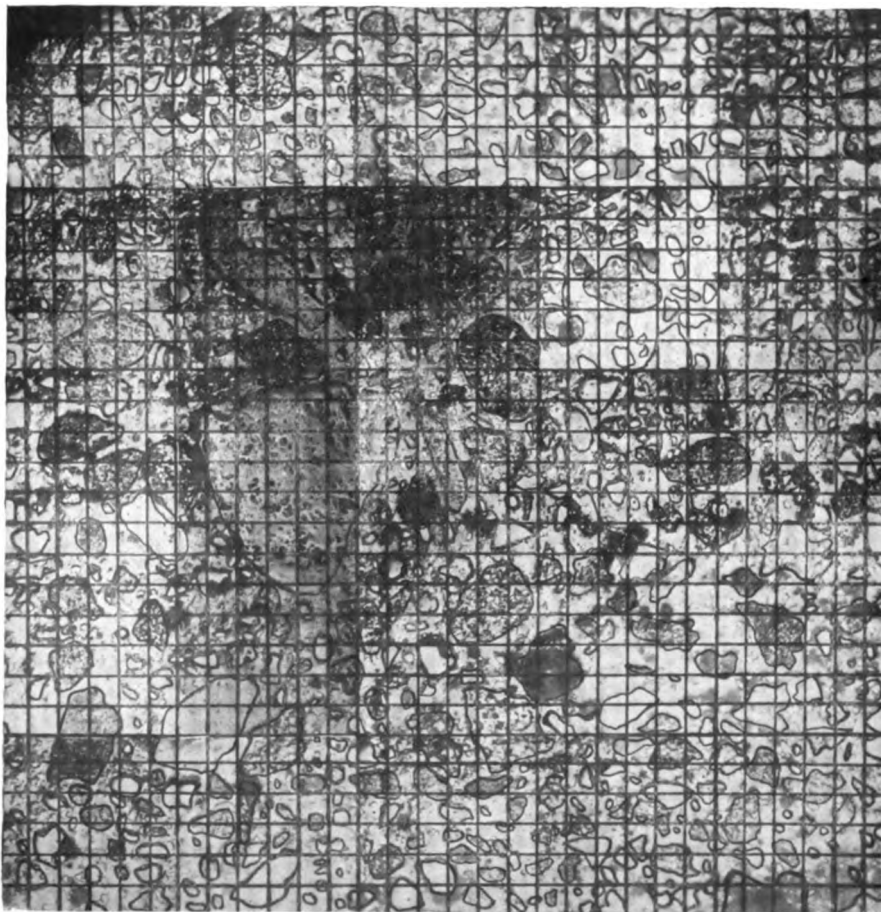


Fig. 20. Composite Photograph for determining Sand Volumes : 23 Diameters.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

DATA NEEDED.

There are now but three other determinations to be made, in order to have full data. These are: (1) the weight of the specimens, large and small, preferably in grams; (2) the volumes of the specimens, large and small, preferably in cubic centimeters, and (3) the specific gravities of the sand and stone, determined from samples dislodged from the mass. The specific gravity of sand and stone can usually be taken as 2.65, thus omitting the exact determination, unless the character of the materials should be such as to make it seem desirable.

The complete data, then, are as follows :

Ratio of volume of stone to [sand + cement + water + air]. Ratio of volume of sand to [cement + water + air].

Weight of large specimen, including stone.

Weight of small specimen of mortar only.

Specific gravity of sand.

FIGURING SAMPLE IN QUESTION.

These are all that are needed. The calculations are as follows, taking sample of mortar from between large aggregates for micro-examination : Weight of sample,

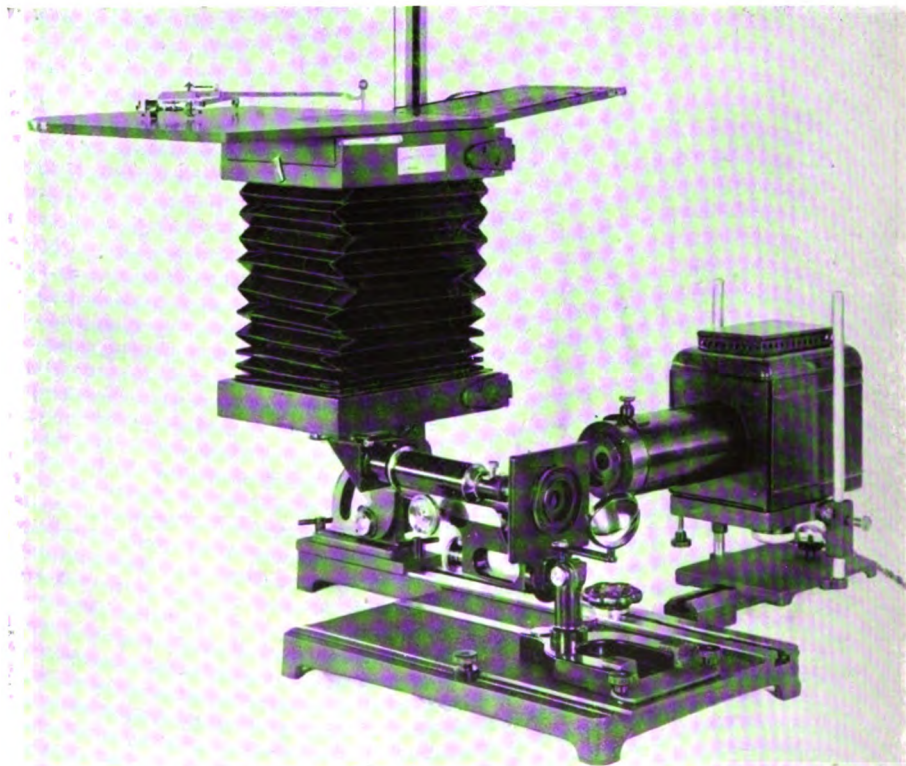


Fig. 21. Microscope and Mounting for Integrating Sand Areas.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

1,680 grams; displacement of sample, 800 cc.; by count, 423 squares are sand, out of a total of 896, or 47.75 per cent. of volume of specimen.

Then $0.4275 \times 800 = 342$ cc. = sand volume; and weight of sand = $342 \times 2.6 = 889$ grams, where 2.6 is the specific gravity of sand.

Therefore the weight of cement, water, and air is $1680 - 889 = 791$ grams; and the volume of cement, water, and air, $800 - 342 = 458$ cc. The weight of entrained air in sample is negligible.

Therefore :

Let x = volume of water and air in sample,
then $458 - x$ = volume of cement in sample, and $(458 - x) \times 3.1$ (sp. gr.) = weight cement in sample,

or, since the specific gravity of water is unity

$$791 - (458 \times) 3 = x$$

$$\text{or } x = 291 \text{ cc.}$$

and $458 - 291 = 167 \text{ cc.} = \text{volume of cement in sample.}$

Moreover, the ratio of sand to cement is expressed by $342 : 167 :: 2.08 : 1$, and the ratio of air and water voids is $291 : 800 = 36.4 \text{ per cent.}$

Turning now to the other data, it has been found that the weight of large sample = 4,625 grams; volume of large sample = 2,100 cc., and, by count, that 67 out of 144 squares are stone. Hence the percentage of stone is $67 : 144 = 46.5 \text{ per cent.}$, or the volume of stone in sample is $0.465 \times 2,100 = 977 \text{ cc.}$; and $2,100 - 977 = 1,123 \text{ cc.} = \text{total volume [sand and cement + water and air] in large sample.}$

But it has been seen above that in 800 cc. of [sand + cement + water + air] there are 342 cc. of sand. Therefore, in 1,123 cc. there are :

$$(1,123/800) \times 342 = 504 \text{ cc. of sand.}$$

Hence the ratio of volume of stone to volume of sand will be $977 : 504 :: 2.01 : 1 = \text{ratio of stone to sand.}$

Therefore, the concrete was mixed in the proportions $1 : 2.08 : (2.01 \times 2.00) = 1 : 2.08 : 4.02$, or, approximately, $1 : 2 : 4$.

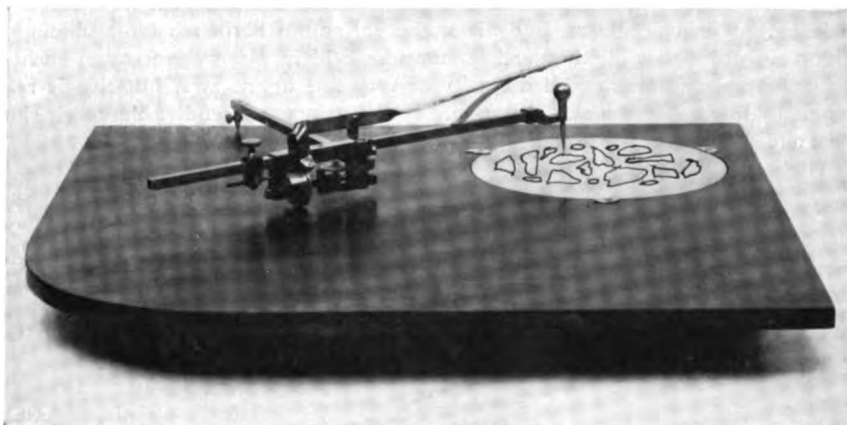


Fig. 22. Special Integrating Table for Measuring Sand Areas.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

PLANIMETER SIMPLIFIES WORK.

In the foregoing demonstration cross-section ruling was employed to give the relative areas of the various aggregates. This method was advanced for the sake of clearness and simplicity, but it is entirely too cumbersome for practical work. A more rapid and accurate way is to integrate the areas of the large aggregate directly from the sections by the use of a planimeter, and to determine the areas of the small aggregate by the same means, in conjunction with the apparatus shown in Fig. 21 (manufactured by Bausch and Lomb).

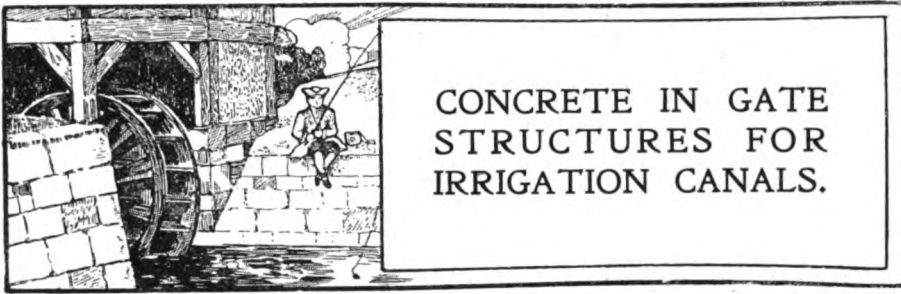
This apparatus, although adapted to many purposes, was designed particularly for cement investigation, with special attachments for the ready performance of concrete analyses. As will be seen, it consists essentially of an ordinary microscope, tipped into the horizontal position; a vertical camera bellows attached to the eye-piece of the microscope through the medium of a prism, and a hooded nitrogen-tungsten incandescent lamp arranged to throw an oblique beam of light on the mortar specimen, held

in place on the microscope stage. By the use of oblique light, colour values are retained in the projected image, so that aggregates can be differentiated from cement.

The special attachment provided for performing concrete analysis is shown in *Fig. 22*. This is placed in position at the upper end of the camera, where the plate-holder, or ground-glass screen is usually inserted. As is evident from the photograph, it is nothing more than a circular ground-glass screen set into an extended wooden table, the surfaces of the two being flush with each other. On this table runs a polar planimeter, by means of which the projected images of the sand grains (sketched in ink for illustration) may have their areas rapidly integrated. A little brake on the register wheel of the planimeter, released by the natural position of the index finger when guiding the tracing point, and applied by raising that finger, prevents slip in moving from one image to another. By means of this simple attachment a series of successive surfaces may be integrated without individual readings, this total area being deducted from the known area of the field circle on the ground-glass, multiplied by the number of fields taken.

WEAK SPECIMENS PRESENT DIFFICULTIES.

Certain difficulties are presented in this analysis by the specimens themselves. As was pointed out in Article 3, some concretes are of such poor quality that the sand grains untouched by cement fall out in the processes of grinding and polishing. When such a concrete is encountered, it is necessary to boil the specimen in a solution of Canada balsam in chloroform, and allow the balsam to harden before grinding and polishing, in order that all the particles may be held in place. Moreover, such concretes usually present the added difficulty of very fine aggregates, but this is readily overcome by increasing the magnification. Such an increase renders the work slower, but has added advantages in reducing the factor of error.



The U.S. Department of Agriculture recently issued an interesting illustrated Bulletin (No. 115) dealing with gate structures generally for irrigation canals, as based on a study of the irrigated regions of the West. Timber, of course, plays a great part in these structures, but the Author, Mr. Fred C. Scobey, also shows that concrete can very frequently be usefully and advantageously applied. We are indebted to the Editor of the Publications Department of the Department of Agriculture for kindly giving us permission to reproduce portions of the Bulletin and also a number of the illustrations.—ED.

Introduction.

THE purpose of all gates considered in this bulletin is the control of the flow of water in ditches or canal systems.

For the most part small and medium-size structures are dealt with. The bulletin should also be of value to directors of mutual water companies, who are themselves irrigators and who are called upon to pass on questions of construction and maintenance.

Materials Used.

A few years ago most of the structures in American canals were of wood, but more recently concrete, both plain and reinforced, has come into common use. Wood has the advantages of cheapness and ease of handling and the disadvantage of rapid depreciation. Concrete has the advantage of permanence, but is costly. The most prominent facts in connection with recent irrigation development under both public and private agencies have been the high cost of water rights and the inability of settlers to make the payments required. In view of these facts there is much to be said in favour of the cheaper wooden structures for original construction with a view to their replacement with more permanent structures of concrete as the wood decays. This will lessen first cost and bring the heavier cost after the lands have been put under cultivation and the expenses incident to the establishment of new farms have been met.

Probably the best practice is to make combined wood and concrete structures, using concrete for the parts which are inaccessible and not easily replaced, and wood for the accessible parts which can be replaced easily.

Radial Gates.

The radial gate is essentially a gate raised and lowered by revolving on a horizontal axle, to which the face is attached by arms centering at the axle. (Fig. 1.)

If it is desired that a radial gate be made comparatively watertight, a strip of rubber belting on each side and at the top may be used. The bottom of the gate may close on a strip of wood set in the concrete floor, or it may close behind a shoulder of wood or concrete.

Paving and Riprap.

The channel immediately above large gate structures may be eroded by the action of racing water, while new banks below nearly all structures become badly washed by the eddies of the issuing stream. The entry channel may be protected by a lining of concrete 3 to 6 in. thick or by a riprap of hand-laid boulders or broken stone. This is sometimes grouted with cement mortar, forming a comparatively smooth, solid surface.

Canals near mountain streams having a bottom paving of boulders may be protected

by the liberal use of these boulders laid as simple riprap or formed into rubble concrete paving by the use of cement mortar. A rough surface, gradually becoming about as smooth as the earth channel below it, will aid the water to secure its "balance" and to pass quietly into the unlined section.

Diversion or River Gates.

As much depends on the diversion gate and it is close to the river, whose performance is always more or less uncertain, it is recommended that a substantial and, if possible, permanent structure be built as soon as possible.

DIVERSION WORKS OF THE SOUTH SAN JOAQUIN AND OAKDALE IRRIGATION DISTRICTS.

The combined structures shown in *Fig. 3* furnish a general idea of arrangement for an efficient method of handling of water at the head of a canal. The joint headworks

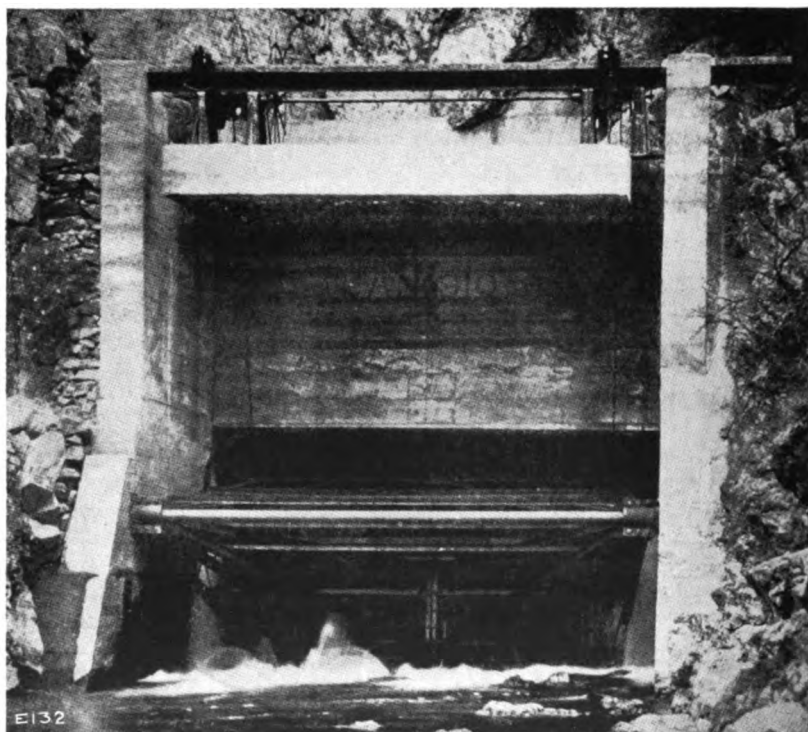


Fig. 1. Radial Gate with Concrete Counterweight.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.

of the South San Joaquin and Oakdale irrigation districts are located in the canyon of the Stanislaus River about 18 m. above Oakdale, Cal.

The complete heading consists of the diversion dam of two arch spans with an intervening buttress, 466 ft. in crest length, and a maximum height of 78 ft., and the joint headworks on the north end of the dam and of the separate Oakdale headworks on the south end.

The joint headworks are built of concrete, part plain and part reinforced, installed upon and against solid rock foundations.

There are four principal elements in the headworks: First, the head wall, with five openings designed to be closed with stop logs in case of accident to the gates below;

water covering the diversion dam more than 3 ft. in depth tops this head wall. Second, a gravity dam placed on a tangent to the curve of the diversion dam, diverging about 16 deg. from a right angle with the centre line of the canal. There are three gate openings at right angles to the line of the canal, each 6 by 9 ft., regulated by massive cast-iron gates raised by screw stems through geared hoists located on top of the gravity dam. The top of the gravity dam is 25 ft. above the crest of the diversion dam, and careful estimates show that maximum flood crests will top the diversion dam about 23 ft. Third, an automatic spillway about 30 ft. long just below the gravity dam at an elevation of $1\frac{1}{2}$ ft. below the crest of the diversion dam, and a sand and waste gate just downstream from the spillway. Fourth, three gates, each 6 ft. wide by $9\frac{1}{2}$ ft. high, set in suitable piers at the head of the joint canal, which shapes up after a transition section about 20 ft. long into a concrete-lined channel 10'33 ft. in total depth, 13'5 ft. wide on

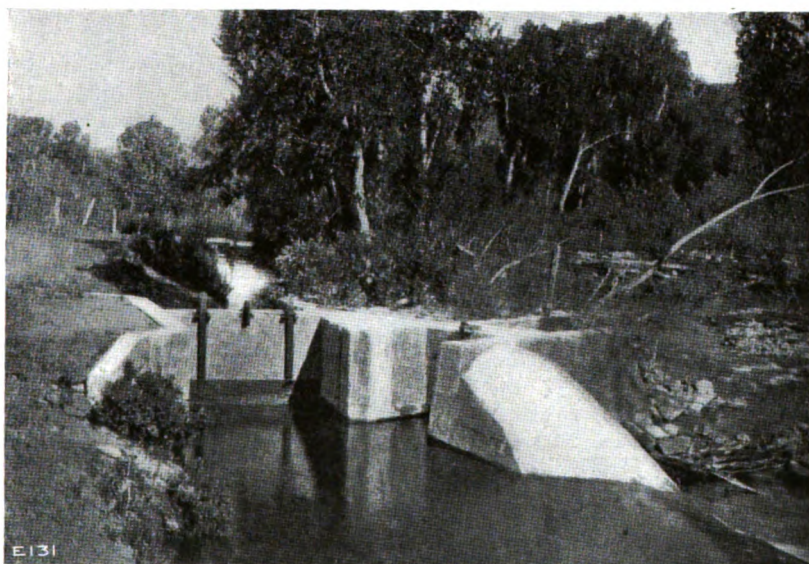


Fig. 2. Diversion Weir, Sandgate, and Headgate of the Jackson Ditch, near Fort Collins, Colorado.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.

the bottom, with side slopes of $\frac{1}{2}$ horizontal to 1 vertical. The designed ultimate capacity of the canal will not be reached until some time in the future when the lower bank is raised and lined about 2 ft. higher.

The floors are made of a 1 : 3 : 6 mixture of cement, sand, and gravel, while most of the balance of the structure is made a 1 : $2\frac{1}{2}$: 5 mixture.

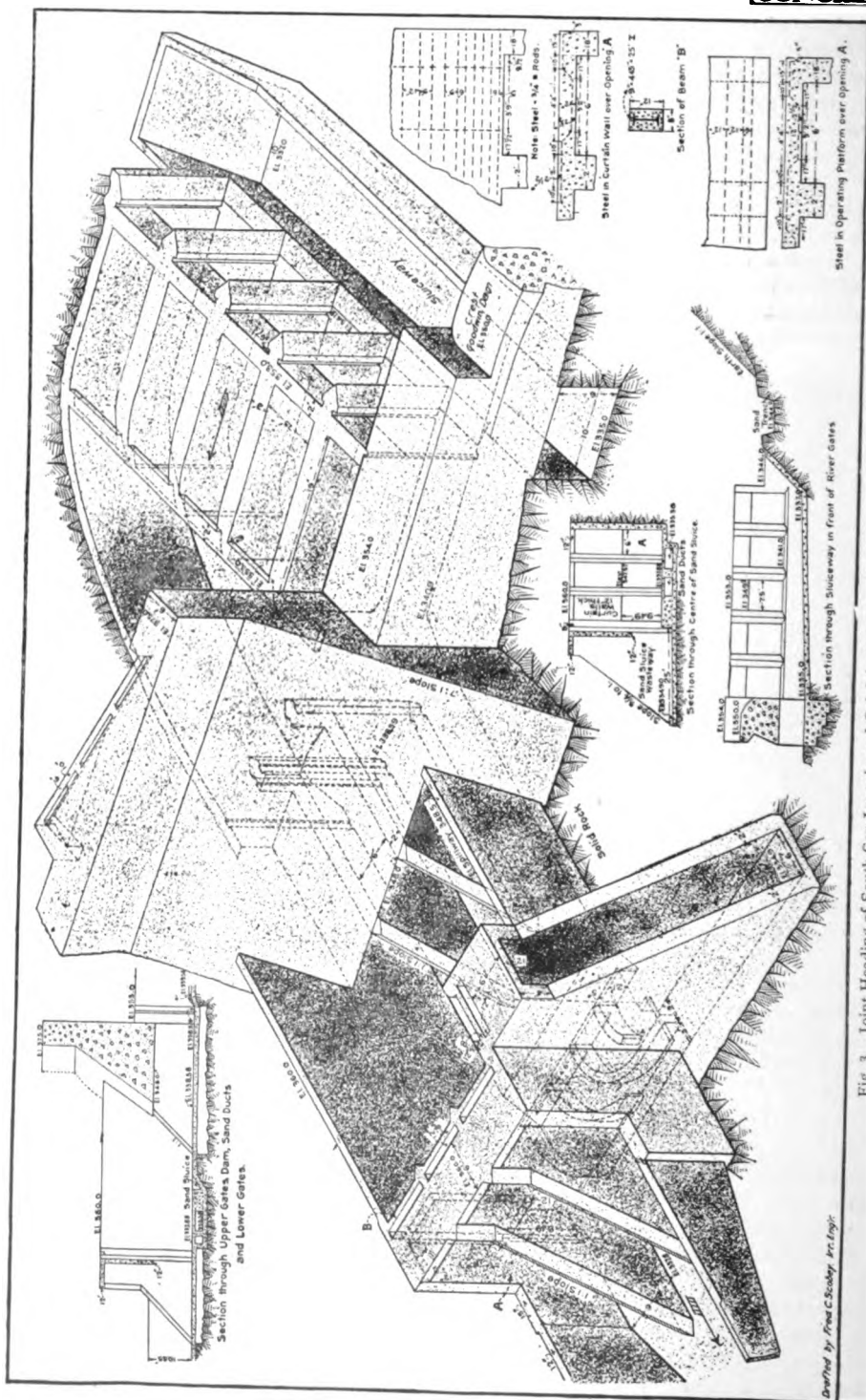
JACKSON DITCH HEADING, COLORADO.

The concrete and steel structure shown in Fig. 2 was constructed 8 m. above Fort Collins, Colo., on the Cache la Poudre River. The gate shutter is of sheet steel operated by double rack stems, connected by a bar which turns both pinions at once. In front of the slots for this gate is a set of auxiliary slots for the insertion of flashboards in case it is necessary to remove the steel gates for repairs during the operating season. The concrete was mixed by hand in a ratio of one part cement to six parts of sand and gravel mixed.

NAPESTA DITCH AND RESERVOIR CO. HEADING, COLORADO.

The heading of the Doyle Arroyo feeder of the Napesta Ditch and Reservoir Co.,

Bul. 115, U. S. Dept. of Agriculture.



Designed by Fred C. Seely, Jr., Eng'r.

Fig. 3. Joint Heading of South San Joaquin and Oakdale Irrigation Districts on Stanislaus River, Cal.
GATE STRUCTURES FOR IRRIGATION CANALS, U. S. A.

located on Doyle Arroyo, in Pueblo County, Colo., is out on the plains 24 m. below Pueblo. (Fig. 4.)

These arroyos in the vicinity of Pueblo are subject to sudden and very violent rushes of water, being dry for months at a time and then carrying a river of water for a short period. The only possible use of such water from an irrigation standpoint is to divert a large head for a short time and store the water in a reservoir. The object of this heading was to divert 850 second-ft. of water from a vertical-sided wash, or arroyo, as it is known locally. The sides and bottom of this wash are in shale where there is little danger of seepage under the structure, but it was necessary to protect the bottom of the wash above and the canal below the gate by a concrete apron, as the water was to be

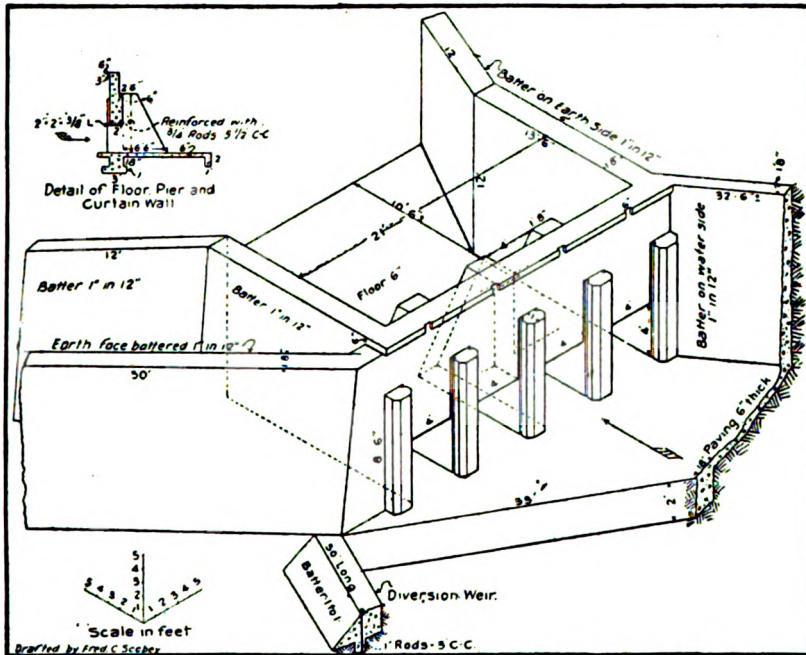


Fig. 4. Headgate, Doyle Arroyo Feeder, Colorado.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.

delivered under a head of several feet, through undershot gates, where the resulting velocity and scour would be very heavy.

It was necessary to build high curtain walls above the openings in order to bulk-head out the surplus waters. The canal below the structure is 20 ft. wide on the bottom, carries water 6 ft. deep, and has side slopes 1 to 1. The structure, as shown, contains 197 cu. yds. of concrete, hand-mixed in a ratio of one part cement to seven parts river gravel taken bank run.

HEADING, THE HIGHLAND DITCH CO., COLORADO.

The heading for the inlet ditch of the Foothills Reservoir, located in Boulder County, Colo., about 10 m. from Longmont, is a good example of a diversion from a stream which is plentifully paved with heavy cobblestones. (Fig. 5.) This paving forms a natural riprap, effectually preventing scouring of the bottom above and below the structure, which eventually would cause the failure of the gate through undermining but for these cobbles.

In order to raise the crest of the diversion dam in times of very low water, a set of

flashboard guides is loosely placed in tin-lined holes in the concrete dam proper, the tin acting as the form for the holes when cast.

This structure, including the bridge in connection, contains 90 yds. of concrete, reinforced with $\frac{5}{8}$ -in. twisted bars. The concrete was machine mixed in a ratio of 1 : 3 : 5 cement, sand, and river gravel.

HEADWORKS, SOUTH BOULDER AND COAL CREEK DITCH, COLORADO.

A short distance above the town of Eldorado Springs, on South Boulder Creek, Colorado, the South Boulder and Coal Creek Ditch diverts 53'55 second-ft. of water.

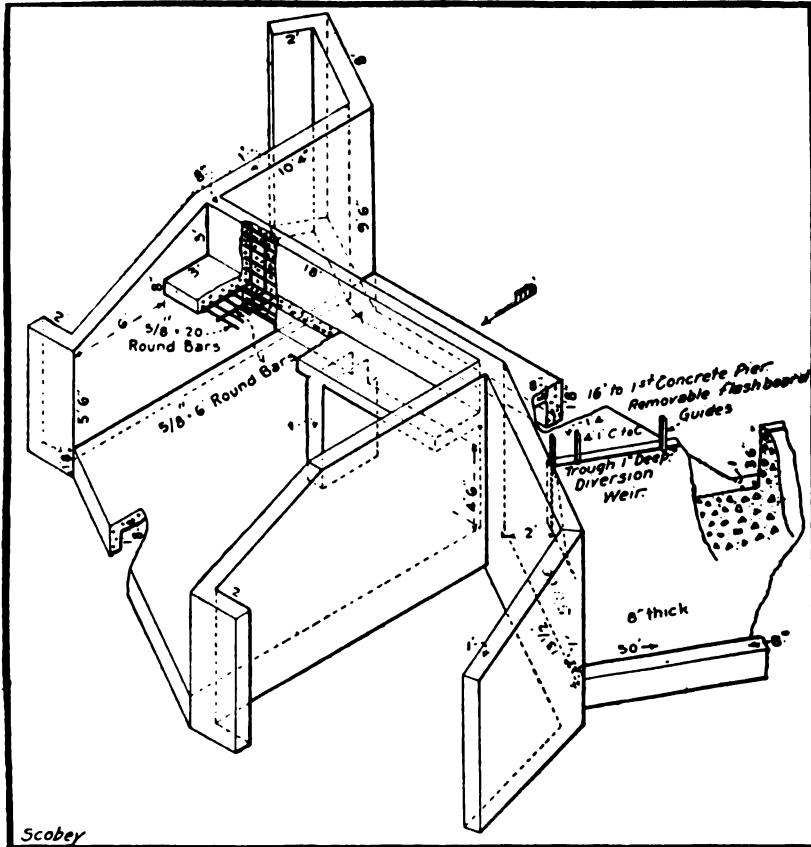


Fig. 5. Concrete Portion of Headgate, Highland Ditch Co., Colorado.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.

As shown in Fig. 6, the creek at this point has a very rapid fall in a canyon. The bottom is strewn with boulders from the size of a cobblestone to that of a small house.

The diversion dam is a makeshift of boards spiked to a heavy cross timber, which is braced against the boulders of the creek, the whole structure being weighted with a sloping pile of loose cobblestones.

The gate structure has 8-in. concrete side walls, 10 ft. long, which bond with the boulders of the canyon, in effect merely squaring up the face of the boulders. Two vents are formed by a central concrete division wall 8 in. thick. The gate shutters are each 2 ft. 4 in. wide, sliding in 2 by 3 in. grooves. The individual boards are of 2-in. stuff firmly bolted to a wide gate stem of the same thickness, which is bound around the

edge securely with an iron strap. Small bolts pass completely through the gate stem edgewise and hold the strap to the stem. The outside gate slots are cast in the concrete,



Fig. 6. Headworks at South Boulder Creek, Colorado.

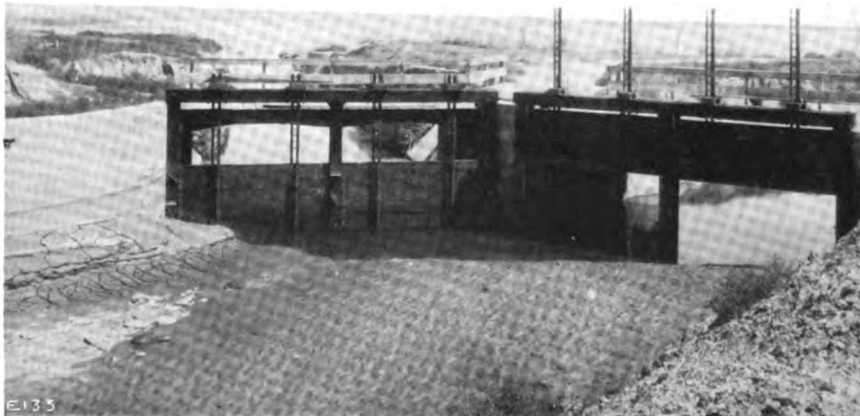


**Fig. 7. Bifurcation Works, Holbrook Irrigation District, Colorado.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.**

while the inside slots are built up of timber secured to the division wall by anchor bolts. The concrete is a mixture of one part cement to six parts sand and gravel found near

the site. The double concrete flume below the shutters is covered with a timber foot-bridge for a trail leading up the canyon.

BIFURCATION GATES, HOLBROOK IRRIGATION DISTRICT, COLORADO.

About 9 m. below the head of the Lake Canal of the Holbrook irrigation district, in Otero County, Colo., the inlet canal to supply Dye Reservoir leaves the main canal. The regulation of water between these two is effected by means of the twin structure

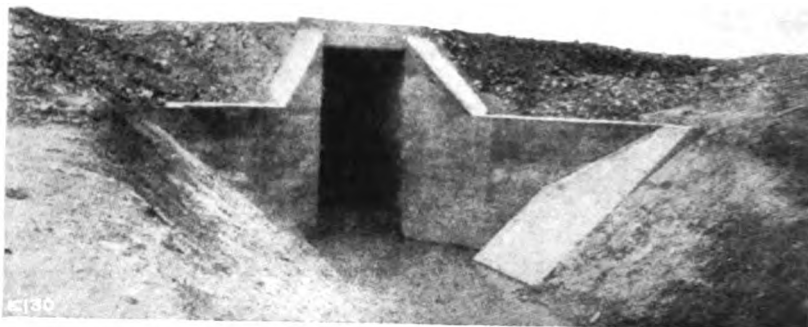


Fig. 8. Headgate, Low Line Lateral, Rock Creek, Wyoming.



Fig. 9. Sand and Waste Gate, Amity Canal, Colorado.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.

shown in Fig. 7. This structure illustrates very well the general practice in Colorado of combining wood and concrete. It is noted that the wings, floor, and side walls are of concrete, while the gate shutters, the front guides, and the whole bridge structure are of wood. As these parts are all bolted to the concrete they may be replaced as they rot out, whereas the portions most difficult of access are made permanent.

The concrete was a 1 : 3 : 5 mixture of cement, sand, and river gravel.

Lateral Headgates.

Lateral headgates divide themselves naturally into two distinct classes—those having essentially a tube of some form through the bank, and those which take an open-box culvert form.

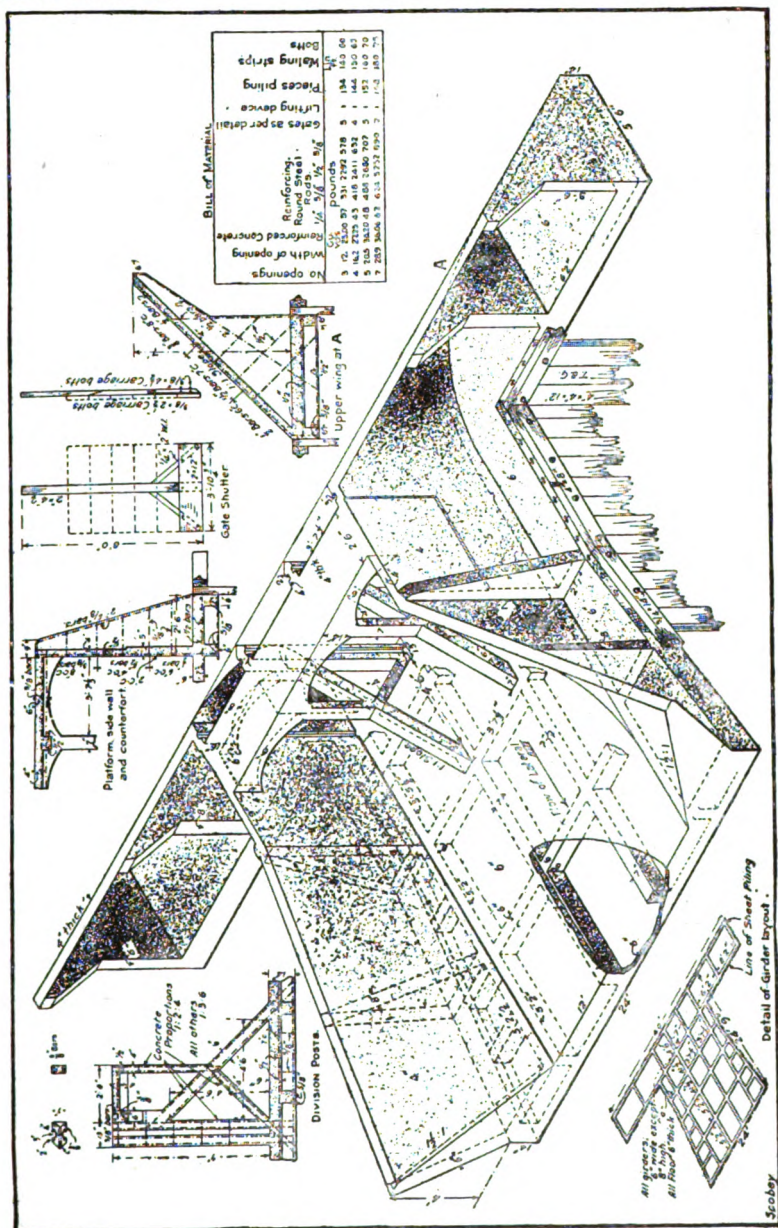


Fig. 10. Branch Canal or Lateral Headgate, California Development Co.
GATE STRUCTURES FOR IRRIGATION CANALS, U S A.

HEADGATE, LOW LINE LATERAL, ROCK CREEK CONSERVATION CO.

The Low Line lateral of the Rock Creek Conservation Co. of Rock River, Wyo., receives water from the Bosler No. 3 ditch. It is 10 m. long, 8 ft. wide on the bottom at the head, and carries water 4 ft. deep on a grade of 2.64 ft. per m.

The structure, built in 1911, at the head of the lateral, also serves as a wagon bridge, *Fig. 8*. It is of concrete with steel gate shutter and lift.

The shutter is placed in a vertical position at the line of the water side of the bank crest. A penstock 5 ft. wide, with side walls 18 in. thick sloping down on the top from the height of the bank to 4 ft. above the floor, leads from the main ditch bottom to the shutter. The floor slants down 2 ft. from the grade line of the ditch to the gate opening. The latter is 3 ft. 7 $\frac{3}{4}$ in. high and 3 ft. 4 in. wide under a curtain wall 1 ft. thick.

The floor extends 15 ft. downstream from the gate shutter and then drops 4 ft. vertically into a water cushion 10 ft. long. At the lower end of the latter a vertical raise of 2 ft. makes the net drop in the structure 2 ft. The side walls through the bank are 5 ft. apart and 18 in. thick, carrying a bridge slab 6 in. thick for a roadway 6 ft.

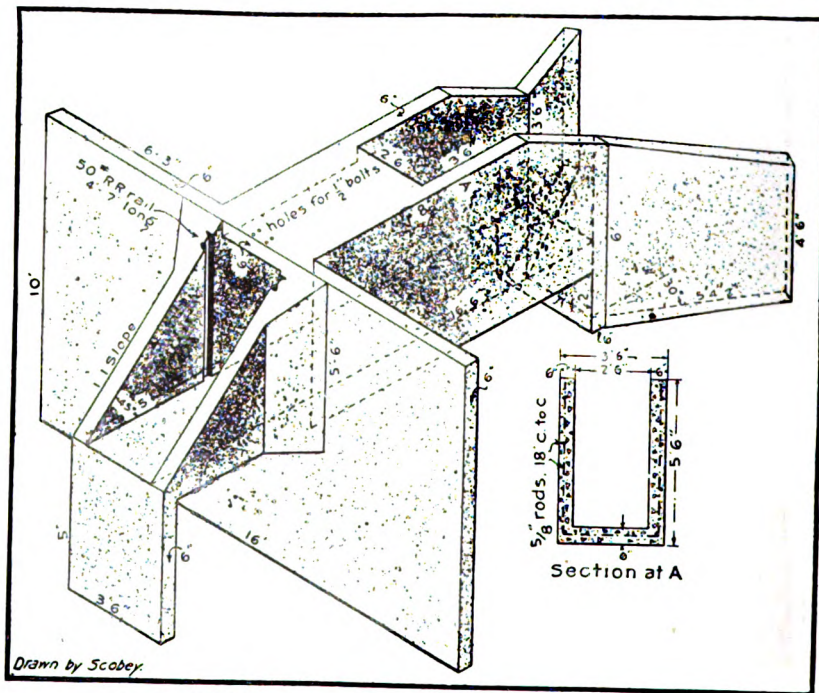


Fig. 11. Reinforced Concrete Delivery Gate, California Development Co.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.

wide. This slab is reinforced the short way with $\frac{1}{2}$ -in. rods placed 6 in. on centres. Below the bridge the lower wings, which are 6 in. thick, slope down from the bank-crest height to 6 ft. above the floor.

This structure contains 111.7 cu. yds. of concrete mixed in a 1 : 2 $\frac{1}{2}$: 5 proportion for plain and 1 : 2 : 4 proportion for reinforced concrete.

LATERAL HEADGATE, CALIFORNIA DEVELOPMENT CO., CALIFORNIA.

The engineers of the California Development Co. have designed standard plans for a reinforced concrete branch canal or lateral headgate. These plans are based on their experience of ten years with wooden structures which these are designed to replace. The plans (*Fig. 10*) show a gate with a minimum amount of concrete, heavily reinforced with steel to give the required strength.

The cut-off walls of this gate are of wooden sheet piling extending 12 ft. into the

bed of the canals. In adapting the plans, light concrete cut-off walls may be used. The girders and counterforts are reinforced to take the tension, thus enabling the intervening slabs to be made much lighter than if the girders and counterforts were omitted.

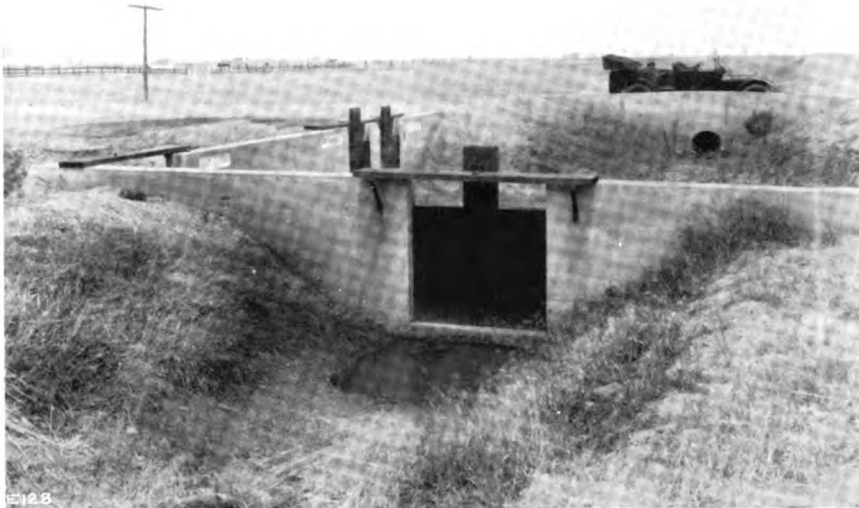


Fig. 12. Combination Check and Delivery Structure, Sacramento Valley Irrigation Co., California.



Fig. 13. Waste Gate, United States Reclamation Service.
GATE STRUCTURES FOR IRRIGATION CANALS, U.S.A.

DELIVERY GATE, CALIFORNIA DEVELOPMENT CO., CALIFORNIA.

Fig. 11 shows a good example of economical design for a reinforced concrete delivery box of the open-culvert form. It is to be noted that the slots for the gate are

placed slightly in front of the foot slab. The reinforcement does not extend down into the cut-off wall, the latter being used solely for the purpose of stopping seepage water.

The 7.6 cu. yds. of concrete is a one to seven proportion of cement and gravel. A small concrete pedestal is bolted in the four holes shown on the top of the operating platform, and to this pedestal is bolted the lever-lifting device for the rack-and-pinion lift.

CHECK GATES, SACRAMENTO VALLEY IRRIGATION CO., CALIFORNIA.

The Sacramento Valley Irrigation Co. has adopted a set of standard plans which may be used in many other parts of the West. The general construction of this gate is adapted either to flashboards or a stem gate. The latter is used most commonly by this company.

This is practically a sheet structure with a small basin attached to the back to withstand the erosion and pound of the falling water. The buttress construction of this basin also braces the front walls.

Fig. 12 shows a combination of check and delivery structures as used by the Sacramento Valley Irrigation Co. Note the inset panel cast in the concrete, upon which the number of the structure is carried.

The concrete mix used is one part cement to three parts sand and five parts broken stone.

SAND AND WASTE GATE, AMITY CANAL, COLORADO.

One of the best examples of the modern construction and use of the "Land" sand gate is the concrete sand and waste gate installed on the Amity Canal by the Arkansas Valley Sugar-Beet and Irrigated Land Co. This structure (Fig. 9) forms an L, one leg of which extends across the canal and acts as a check gate, while the other leg contains the openings of the sand ducts at the bottom of one bay, and also serves to waste excess water by raising the radial gates above the upper cover to the sand ducts.

The complete structure, comprising both the sand and the check gates, contain 135 cu. yds. of plain concrete and 95 cu. yds. of reinforced concrete.

Waste Gates.—Uses of Waste Gates.

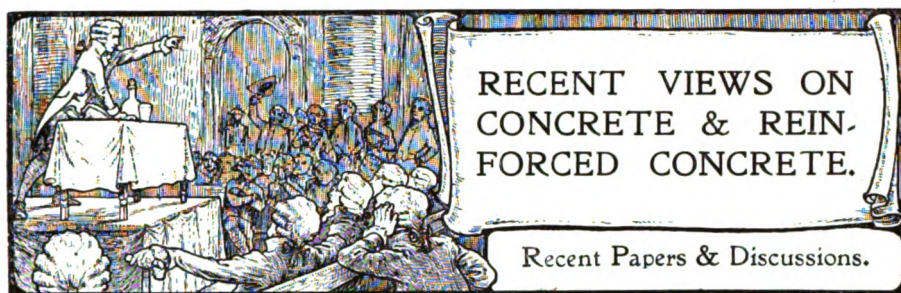
The waste gate is used on canal systems for two distinct purposes: First, it may be simply a safety valve, carrying off excess water which enters the canal during storms or which is turned back into the canal unexpectedly by consumers. Second, it may be located near the head of the canal with a view of running excess water from the river, relieving the pressure on the headgate in times of flood, and reducing maintenance charges by using the large volume of water to develop a high velocity in the upper reaches of the canal, scouring out sediment. It then becomes a combination of waste and sand gate.

There are two general types of wastes. The first comprises those acting purely as spillways, having a crest height deemed the controlling elevation of the safe high-water line of the canal. Excess water reaching the waste tops the crest wall and is carried off in a natural watercourse or an auxiliary ditch constructed for the purpose.

The second type of gate has an adjustable opening extending down to or below the grade line of the canal, permitting all of the water to be turned out if necessary.

WASTE GATES, YAKIMA PROJECT, UNITED STATES RECLAMATION SERVICE, WASHINGTON.

Fig. 13 shows an easily adaptable waste way for small laterals or ditches subject to sudden increases of water. Where the ditch crosses a natural drainage way a concrete waterway is set in the levee on the lower side and the levee up and down stream for a short distance is paved with rubble laid in cement.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

INSTITUTION OF MUNICIPAL AND COUNTY ENGINEERS. THE PROTECTION OF ANCIENT BUILDINGS.

By W. A. FORSYTH, F.R.I.B.A.

At the annual meeting of the above Institution, held last month, a very interesting Paper was read by Mr. W. A. Forsyth on the Protection of Ancient Buildings. We are unable to reproduce the whole Paper, but give below a short extract from the introductory portion of the Paper and the remarks on grouting, as also an extract from the portion dealing with masonry.

THE careful use of an old building ensures its best protection. Municipal and county authorities are frequently the guardians of buildings of historic interest and architectural beauty. The repairs are placed invariably in the hands of the official engineer or surveyor, whose skill in the solution and execution of scientific problems is supposed to fit him for the subtle work of repairing ancient structures. The actual reparations are made by the Works Department, whose experience in highly technical affairs is unquestionable, but whose knowledge of materials and their application for such a purpose is limited and cannot be expected to be otherwise.

Such were the circumstances of a recent case in which one of the Borough Councils in London dealt with the preservation of an historic building in an advanced state of decay. The completed work aroused criticism, and, in the course of discussion, the borough surveyor admitted that the views expressed by an architect, a specialist in this type of work, would have had more satisfactory results, and the particular character—historical and æsthetic—of the fabric would have been more carefully preserved, had such process of repair been generally known. Arising from that incident, a suggestion was made that some general notes upon the subject should be laid before the Institution embodying the views held by the Society for the Protection of Ancient Buildings.

It is not possible to bring the whole or any large part of the subject into review within the limits of this paper. A few general practices can, therefore, only be attempted, and it is hoped that something that is new may reach many whose activities extend to reparations, and who will be induced to apply these recommendations to such structures as town halls, castles, bridges, town walls, gateways, memorials, etc. Old buildings are decreasing in number year by year, so that it is the more important that not only shall they be preserved, but that they shall be cared for and repaired in such a way that their traditional character shall be maintained.

All repairs or replacements should be distinctive, yet harmonious, for they form part of the story in stones. If the truth of the story is veiled by attempts at imitative reproduction, the values of the building are lost.

It is sincerely to be hoped that when the time comes to reinstate the Belgian and French cathedrals and town halls, the reparation will be clearly expressed for all time. It is our duty to history unmistakably to express what is mediæval and what is twentieth-century craftsmanship.

It is important that such structural works as underpinning be not too hurriedly decided upon and carried out. Ample test and observation must be made, for it is known that many a building was safer before than after that operation.

Materials used in repairs should, as far as possible, be new, and local traditional custom should influence their choice and method of building. The use of old materials from other buildings is misleading, and tends to create an undesirable market in such commodities. All old materials, however, such as glass, metal, etc., must be carefully collected and stored.

The tendency to over-repair should carefully be avoided: anticipated defects must not be considered, as a great deal of wanton injury will thereby be perpetrated.

Above all it must be remembered that repairs cannot be hastened; slow-setting materials are invariably those of greater lasting quality.

Any additions that may become necessary should be designed in a different manner from the old buildings to which they are to be attached. They need not be unsightly, although possessing a modern character, so that no mystery need be cast upon the structural story of the fabric.

GROUTING.

Repairs are chiefly required for old walls more than any other features. These have invariably become defective from their faulty bonding, from roof thrusts, or weak foundations. The solidifying of the walls is usually necessary before such operations as underpinning, the insertion of damp courses, the provision of tie-rods, etc., can be commenced.

No greater advance has been made in the repairing processes in recent years than in that of grouting. Many a condemned building has been saved and many a weak structure made sound by this means, and more particularly when used under pressure.

The injurious effects of vibrations upon ancient buildings, especially those having rubble-built walls, due to fast and heavy modern traffic, is a matter of grave importance.

It frequently happens that a fractured rubble wall, a weak footing, an insecure bridge has failed, and can only be repaired by first consolidating the masonry before foundations or other repairs are dealt with. Briefly stated, the method of applying cement grout under pressure is as follows: The operation, like underpinning, must be performed in small sections. All loose, dry mortar and dust is first removed; that which escapes being raked from the open joints is blown out. Great care must be taken not to dislodge the aggregate of the wall—that is to say, the rubble, the bricks or stones, as the case may be. Large facing stones should be wedged up.

The parts of the wall thus cleared should then be well saturated, also under pressure, after the face joints are blocked up, usually by wet clay. This moistening will slake the inert properties of ancient lime, and assist the process of solidification. The grout, mixed according to requirements, is then forced by the machine into the wall through a small aperture left for the purpose. The clay joints should be watched, in case the grout escapes prematurely; experience will determine when the particular section is fully grouted.

It is usual to use slurry of one part of cement to one part of sharp, but not too coarse, sand. Frequently neat cement is injected into the fissures. The effect of these injections is to solidify the wall, leaving it in a state reasonably safe for the underpinning, the cutting out of cracks, and bonding in strong material, or other operation.

The clay sealing of the joints is removed after ordinary setting has begun to take place, when it will usually be found that the grout has fully penetrated the wall or structure under treatment. It is then desirable to point the more open joints of the external face to exclude weather. It is not necessary, however, to point internal joints, but to leave them open in order to show that the old materials have been grouted. Grouting by hand has its limitations, and is dependent upon gravitation. Under mechanical pressure the liquid rises to considerable heights and travels surprising distances.

There are occasions—usually in thick walls—when hydraulic blue lias lime-mortar would be adequate as a grouting medium. Expansions and, in many stones, an unpleasant discoloration are then avoided.

Lime grout is very desirable where walls have not fractured but have hollow places. Lime-mortar built walls have endured a sufficient time for all ordinary purposes. There are many limes which harden to a very great degree. Smeaton used Leicestershire limes in the erection of lighthouses. One of the Derbyshire lime-works has been working continuously for seven hundred years.

A great advantage of this method of repair is that the two faces of a wall are maintained during repairs.

Winchester Cathedral has been saved by grouting. Sir Francis Fox has also grouted and saved historic bridges condemned by county authorities; he has so treated the walls of a large institution when the entire rebuilding was threatening the trustees, whose finances could not bear the strain; he has grouted the walls of an early eighteenth-century London building upon which a dangerous structure notice had been served by a watchful district surveyor, who, after testing the repaired house, accepted the process for the more customary one of demolition.

Grouting would materially have contributed to the retention of Boswell's House in Great Queen Street. The demolition of this interesting building is a very serious loss to students of architecture. It was by no means beyond repair, and was a valuable example of early design in street fronts.

Many a condemned external or party wall is pulled down when grouting and bonding in of new work would have rendered the structure safe.

MASONRY.

The superficial decay of stonework is largely due to atmospheric causes, which are usually more aggressive in the neighbourhood of large towns.

No moulded or carved feature should be on any account reproduced either in stone or any other material. The Society for the Protection of Ancient Buildings has always recommended that badly decayed ashlar be rebuilt in roofing tiles laid horizontally and rendered in mortar.

The same methods are applicable to brick walls, and it has the great advantage of ensuring the pinning up of the new material where such is not always possible with the insertion of large stones or bricks.

When a weak wall cannot be grouted, and it is only possible to repair by bonding in new material, it is important that the more interesting face be retained at the expense of the less valuable.

Rubble facing which has become loose should be repaired by re-using the old stones or flints, cleaning and mixing them with lias lime-mortar stiffened by a small proportion of cement, and the whole laid in and punned up to a shutter board, after the manner of concrete building.

In less decayed rubble facing the loose members may be reset and pinned in with a roofing tile. This was a mediæval practice.

In the repair of masonry and brickwork reinforced concrete can be structurally used with advantage in footings, floors, and lintols. The proportion of steel to concrete must not be unduly large, but sufficient for taking up tensile strains. The metal must be well covered by the concrete for preservation reasons.

NEW ENGLAND ASSOCIATION OF GAS ENGINEERS.

A CONCRETE PILE HOLDER FOUNDATION.

By FRANCIS E. DRAKE.

Extract from a Paper written for the forty-fifth annual meeting of the New England Association of Gas Engineers.

ORIGINALLY all foundations were carried down to hardpan, but I believe some 2,000 years ago the first wooden piles were used. From that day on, piles became more and more common and served their purpose well for supporting structures built upon land which was of uncertain qualities, providing they were cut off below the water-line and always kept submerged. With the advent of concrete this was no longer necessary, for concrete was found to be practically indestructible when mixed of good materials and then properly formed and set.

Thus, when the contractors, from whom we ordered our 5,000,000 cu. ft. holder, came to settle the question of foundation, it resolved itself into one of concrete piles, because of two reasons: first, durability; second, cost.

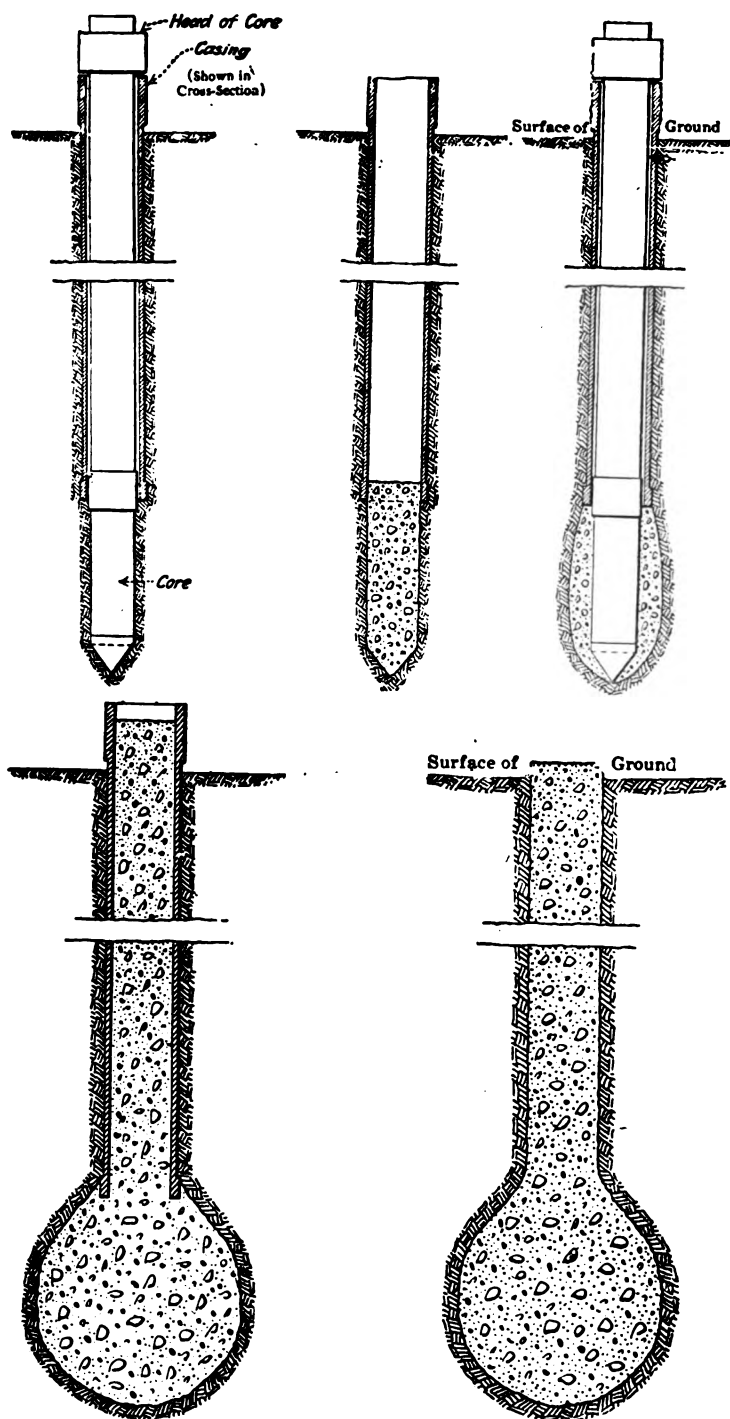


FIG. 1. METHOD OF FORMING PILE.

The holder when completed will have a total height of 211 ft. and a diameter of 204 ft. 6 in. There will be three 30-in. and one 20-in. inlet and outlet pipes, and the holder when inflated with air will throw a maximum pressure of 12.76 in.

Twenty-five or thirty years ago the site of the new holder was some 12 or 14 ft. lower than it is to-day, and this lower stratum was marshland. This was pretty well known, but, to be safe, borings were made by driving $\frac{3}{4}$ -in. extra heavy pipe. It was found by these tests that there was approximately 10 ft. of fill, composed of almost everything from ashes and clinker to brickbats and old fire clay materials. Below this fill was found a hard layer of peat about 12 to 18 in. thick, and then was perhaps a foot of soft muck, below which was found good hard sand mixed with clay. These determinations were corroborated when the piles were driven, for in every instance the piles "brought up" in the hard peat at a depth of from 10 to 11 ft., and, after forcing through this peat and then the soft muck, "brought up" in good hardpan at an average depth of 13 ft. 9 in.

The pile as driven is known as a "Pedestal Pile" and its method of driving and formation is rather novel. After having determined, by means of test pipes, the probable depth to which the proposed piles must be driven, a cylindrical steel casing, 16 in. in diameter by $\frac{3}{8}$ in. thick, with outside reinforcing bands top and bottom, and of the required length, is used as a form for the pile. Inside of this casing is fitted a core, which is a smaller and longer pipe with a cast steel point and an enlarged cast steel head, the latter engaging the top of the casing, while the former projects some 4 or 5 ft. below the casing. In the head of the core is fitted an oak driving block, which receives the blows of the hammer.

For forming the pile the core is fitted into the casing and both are driven into the ground by means of a 3,000-lb. steam hammer, which has a drop of 36 in. and delivers sixty-eight blows per minute. The casing and core are driven to a refusal of eight to ten blows per in. The core is then withdrawn and the resulting void partly filled with a charge of concrete.

After the concrete has been charged into the casing, the core is then lowered and used as a ram, driving down through the charge, which is in this manner compressed and forced out against the surrounding soil, pushing back and compacting the earth. The rammer is now withdrawn, more concrete shovelled in, and the ramming process again repeated, causing the concrete to be still further forced out into the surrounding earth. This ramming operation is carried to a refusal of about fifteen blows per in., at which time the charge has been forced into the surrounding soil, so that there is a base formed about 3 ft. in diameter. The ram is then removed and the casing filled to the top with wet concrete. The casing is now pulled up slowly and evenly, with the ram spotted on the concrete holding it in position. With the removal of the casing the concrete falls into position, completely filling the thin space formerly occupied by the shell. With the withdrawal of the casing, it will be found that the surface of the concrete will have sunk some 3 to 6 ft., according to the length of the pile. The contractors claim that the volume of shrinkage conforms exactly with the volume of the casing wall which it replaces. The resultant pile shows a perfect concrete stem 17 in. in diameter surmounting a bulbular base approximately 3 ft. in diameter.

Fig. 1 shows a diagrammatic sketch of the method of forming the "Pedestal Pile." An excellent footing course is formed, and in the case of our foundation, with piles spaced on 5 ft. 2 in. centres, there must be practically a continuous footing under the whole foundation. Fig. 2, is a detail working drawing of the foundation as built at Lynn. It is 204 ft. 6 in. in diameter, and the piles being on 5 ft. 2 in. centres make a total number of 1,361. As will be noticed from the drawing, the piles are brought to grade with the exception of the outer row, which is left 3 ft. below the surface to receive an outer ring of concrete 2 ft. wide by 4 ft. deep. This outer ring stiffens the slab where the steel tank superstructure rests, and also goes into the ground far enough to prevent any lifting action from frost.

After the piles were completed, two $\frac{3}{4}$ -in. round rods, for the purposes of reinforcing, were placed across the top of each line in both directions. Between these $\frac{3}{4}$ -in. rods and directly on top of them, extending in both directions, were placed $\frac{1}{2}$ -in. rods; the latter being wired directly both to the $\frac{3}{4}$ -in. rods and to themselves. After the reinforcing was completed the piles were ready to receive the pad, which was about 12 in. thick. Before the pad was poured, the two piles which brought up with the easiest penetration were picked for test.

The concrete as used in piles and pad was a 1 : 2 : 4 mixture. There was used 5,479,075 lb. of crushed stone, one-half of which was used in the piles and the other half in the pad. The stone for the piles was mixed so that about two-thirds was $\frac{3}{4}$ in. and one-third was $1\frac{1}{2}$ in., while for the pad, approximately two-thirds of the stone used was $1\frac{1}{2}$ in. and one-third was $\frac{3}{4}$ in. The amount of sand used was 1,490 tons, while the cement totalled 27 cars, or 16,200 bags. For reinforcing the pad, 54,870 lb. of $\frac{1}{2}$ -in.

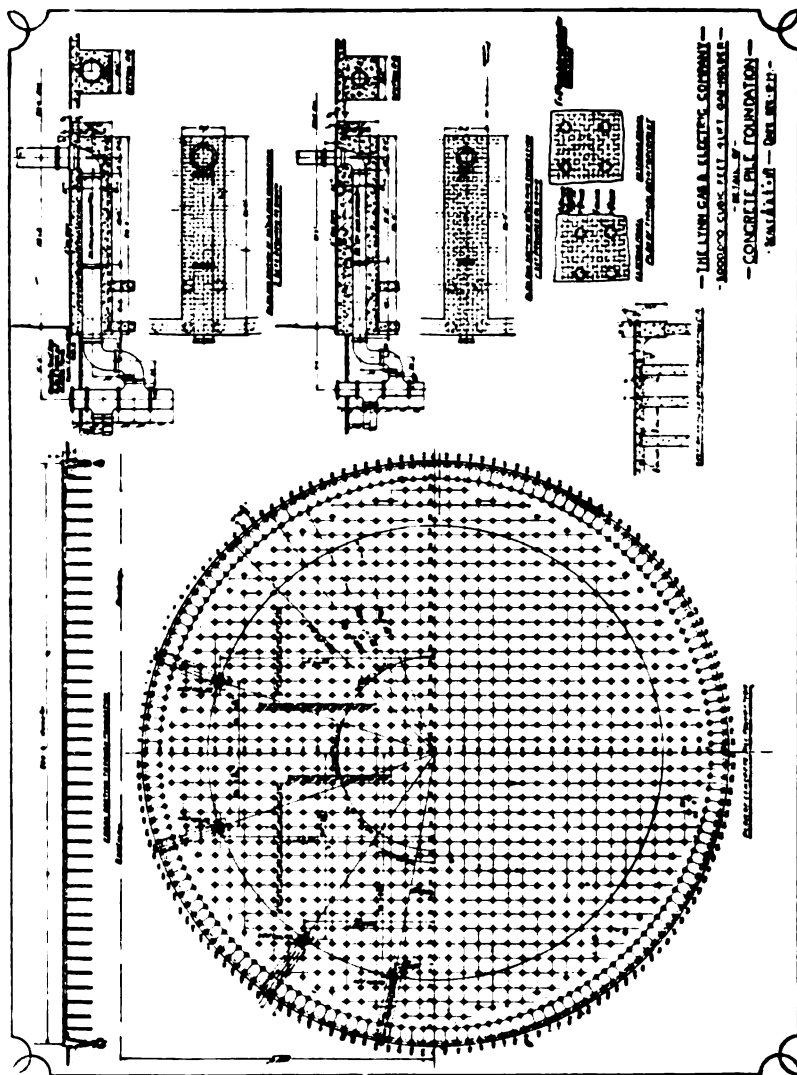


FIG. 2. DETAIL DRAWING OF FOUNDATION.

rods and 45,913 lb. of $\frac{3}{4}$ -in. rods were used. The total lineal feet of piles driven was 18,730. The longest pile was 15 ft. 6 in. and the shortest, outside of those of the outer ring, was 12 ft. 6 in. Those in the outer row were left below grade and averaged about 11 ft. 6 in. long. The average length of all piles was 13 ft. 9 in., and the average number driven per day was 52.4. This was with two drivers, and gave an average of 20.6 minutes for the complete formation of each pile. The entire foundation required 41 working days to construct.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

EXTENSION AND ALTERATIONS OF THE WIGAN BATHS, MILLGATE.

Second Class Swimming Bath.—The scheme included taking down the existing walls at the west end of the second class swimming bath. The pond has been extended by 7 ft. and a sloping stopped gallery has been erected extending from the floor level to a height of 16 ft.; the gallery is constructed of reinforced concrete and has seating accommodation for 200. This gallery will be utilised for dressing purposes when the baths are crowded, and for seating when swimming galas are held. Under the gallery ladies' and gentlemen's dressing rooms, each 15 ft. by 13 ft. have been constructed, with a w.c. to each and urinals in the gentlemen's; the upper portion of the walls are red pressed brick and the lower portion have a glazed brick dado. There is also provision for two store rooms where there is not sufficient height for the underside of the gallery to be utilised for other purposes. A new entrance has been provided from College Street: this is 7 ft. wide and has white glazed tiled walls, vestibule doors with leaded lights. The roof over the extended portion of the swimming bath has a deeper lantern than the old one and is fitted with swivel windows to assist ventilation.

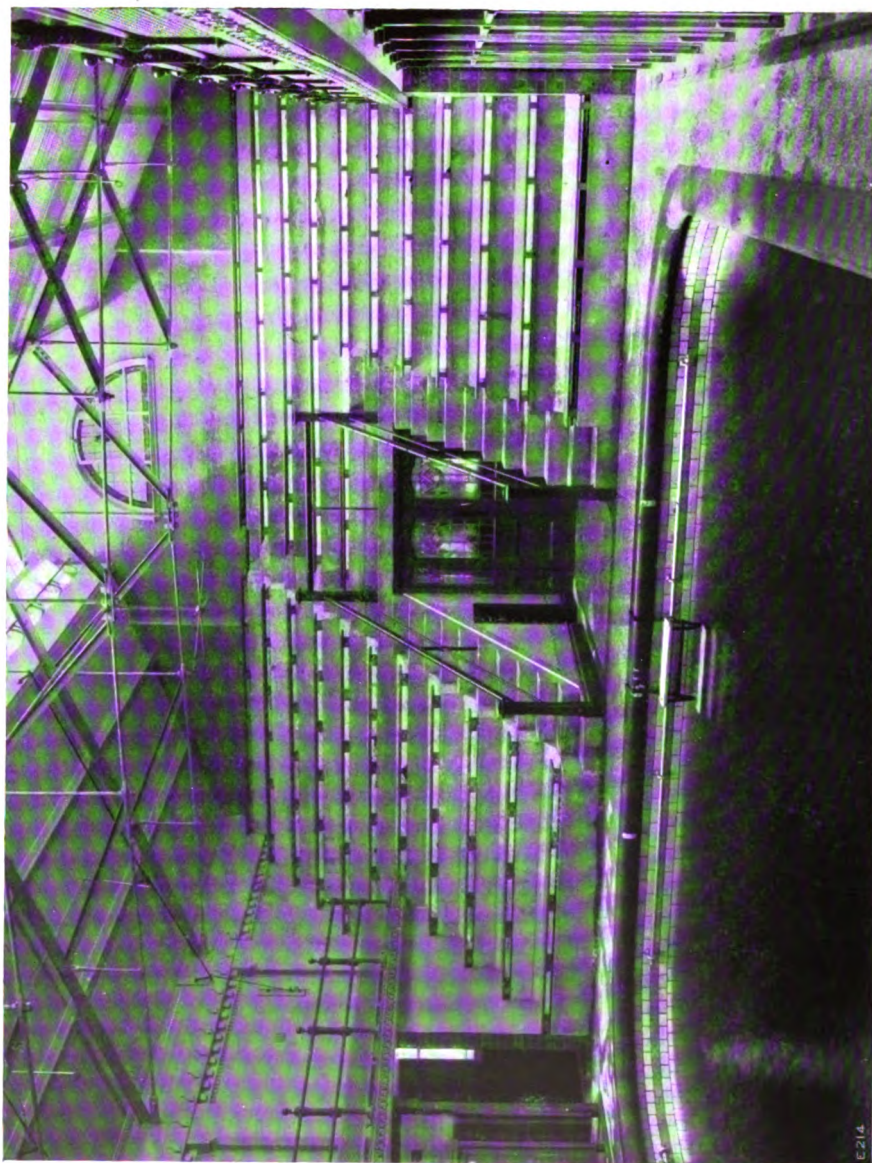
A new men's washing room has been constructed which is 19 ft. by 13 ft.;



Interior of Slipper Baths, showing reinforced concrete roofs and floors.
EXTENSION AND ALTERATIONS OF THE WIGAN BATHS, MILLGATE.

it is fitted with four shower baths and a needle bath, and a urinal and w.c.'s; the walls are faced with white glazed bricks, and a lantern light constructed in the roof with swivel windows for ventilating purposes.

Slipper Baths.—The old slipper baths were very much out of date with their



Interior of Plunge Bath, showing reinforced concrete step gallery.
EXTENSION AND ALTERATIONS OF THE WIGAN BATHS, MILLGATE.

wooden floors and partitions and lack of light and ventilation; the old iron baths, which were cased in woodwork, the same were getting in an insanitary condition. The partitions, floors, walls, and roofs to the old slipper baths have been taken down and a new reinforced concrete roof with lantern light the full length of the baths has been

constructed; the same has swivel windows for ventilating purposes and the floors have been carried out in reinforced concrete and finished in terrazzo. The partition walls have been constructed with Shepwood white glazed brick, 2½ in. thick, and the existing walls finished with glazed tiles to match the partitions, and modern white glazed porcelain baths have been provided with showers to the first class bath.

Boilers and Heating.—The whole of the heating arrangements to the baths have been altered, and the old boilers, which had been used for thirty-three years, have been taken out and two new Cornish boilers, 18 ft. by 6 ft., have been provided. New calorifiers with copper coils, cylinders and pipes, for providing hot water and heating, have been provided.

Drainage.—The whole of the drainage arrangements in the baths have been altered and separate drains have been provided.

The whole of the above work has been carried out to plans, specifications, and quantities prepared by the Borough Engineer, Mr. A. T. Gooseman.

The contractors for the buildings were Messrs. W. J. Bickerstaff, Wallgate, Wigan, who also carried out the reinforced concrete work. The reinforced steelwork was provided by Messrs. Johnson, Clapham and Morris, Ltd., Manchester. The contractors for the heating, hot water, and calorifiers were Messrs. Thos. Bradford and Co., Crescent Iron Works, Salford, and Messrs. Farrimond and Glover, Ormskirk Road, Pemberton. The contractors for the boilers were Messrs. Tetlow Bros., Bottoms Ironworks, Hollinwood.

A LARGE CONCRETE ARCH CULVERT.

THE illustration here shown is of a reinforced concrete arch culvert over the Lackawanna River, Forest City, Pa. It has a 42 ft. span with a barrel 112 ft. long with spandrels and wing walls to accommodate the proposed fill for the elevated roadbed.



A LARGE CONCRETE ARCH CULVERT.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

Design of Steel Bridges. By F. C. Kunz, C.E.

Published by The McGraw-Hill Book Company,
6 Boulevard Street, London, E.C. Price 21/- net.

The preface of this volume states that it is the first of four, each complete in itself, which the author contemplates; and from a study of the first they should form a useful series.

The author states that his aim has been to give the bridge engineer numerical examples and results of the best modern practice in the designing and estimating of steel bridges, and, although the book naturally represents American practice, the fundamental principles are the same in all countries, and it will be worth study by British engineering students and form a useful reference book for qualified engineers in this country.

The methods of calculation and design are very clearly demonstrated, and are rendered easy to follow in all cases by numerical examples showing the application of the formulæ, and plates and diagrams to elucidate the principles of design.

The methods of graphic calculation by the use of influence lines are recommended, and the value of their employment is clearly demonstrated. The special loadings given, and used in the preparation of many of the tables, are those met with in American practice, being the "Cooper E." loadings in the case of railway bridges and heavy cars in the case of highway bridges, but the tables can be readily adapted to other loadings by the use of the methods given for their compilation by the author.

The skeleton diagrams giving the stress coefficients for the members of trusses with parallel and polygonal chords under uniformly distributed loading will be found useful for facilitating the calculation of the stresses.

The methods used for the calculation of the stresses in the sub-members of deep trusses are perhaps more interesting than useful to English engineers, whose practice seldom requires the design of bridges with the very deep trusses so common in the United States.

The calculation of the stresses in and the design of lateral bracing, portals, and braced piers is dealt with, and some space is given to the treatment of the elevated railways and movable bridges.

Arched and cantilever bridges are each given a chapter, showing the methods adopted in calculating the stresses and the features essential to good design.

The requisite details of riveted and pin connections for the members of trusses are given, and useful hints will be found in the portion of the book dealing with these subjects.

An appendix is added, which includes the properties of various steel sections, many useful tables, and General Specifications for Steel Railway Bridges, compiled by the American Railway Engineering Association.

The series of folding plates at the end of the book giving skeleton diagrams and details of the design of actual bridges is a valuable addition to the contents.

The National Physical Laboratory—Report for the year 1914-15.

Teddington: W. F. Parrott, The Causeway.

This Report, which has recently been published, is interesting as showing the amount of useful work that has been accomplished during the past year in spite of the fact that the war has considerably hampered operations, no less than fifty of the staff being on service. A good deal of advice and assistance have been given to the Admiralty and War Office, and it may be said that in addition to the aeronautics work, the time of a number of the senior members of the staff has been given almost entirely to war research.

Dr. Stanton's report on the Engineering Department contains an interesting record of useful work, and deals with a number of researches. The investigation of the resistance of materials to alternating shear and combined stresses has been undertaken as part of the general scheme of investigation of the British Association Committee into "The More Complex Stress Distributions in Engineering Materials," and it is hoped that the first part of the research

will be completed before the meeting of the Association this year. The methods of notched bar impact testing formed the subject of research, and the work in connection with this is now proceeding. The experiments on the high-level girders of the Tower Bridge, undertaken in connection with the subject of wind pressure on structures, have been continued through the year, and the apparatus is to remain in position for the present in the hope that some records may be obtained with winds of storm force. The comparison of the various tests for determining the hardness and wear of metals was commenced, and certain difficulties were met with which it is hoped will be overcome in the future. One of the most important matters put in hand is that dealing with the strength of reinforced floors and columns, and a special machine is now under construction, which has been presented to the Laboratory by Sir John Cowan. It is hoped that the machine will be completed in the autumn of the present year, and it will then be possible to test floors 16 ft. by 6 ft., and columns up to 14 ft. high. The result of this research work will be largely awaited by constructional engineers.

The Report is presented in a very good manner and contains a great deal of interesting matter.

Recapture and Expansion of British Trade —The Building Trade.

Published by The International Correspondence Schools
Ltd., Kingsway, London, W.C.

This is an excellent booklet which sets forth in a very convincing manner the need for energy and organisation by us in this country if we are to capture a large portion of the trade that will result after this war is over, more especially in the building industry. The subject is dealt with generally in the first instance, and the outlook and opportunity are brought to the reader's notice, and the specialist trades are then discussed from all aspects. Attention is called to the lack of organisation and proper training in the country during past years, and figures are quoted to show how the commerce of Germany has been developed until we are almost hopelessly behind. Some sensible remarks are made

with regard to the need for improved buildings if British manufacturers are to successfully compete with the Germans, and it is necessary for architects and engineers to realise this fact and study modern factory design and construction in order that they may be competent to do their part towards the realisation of this improvement, and much can be learnt from Germany and America in this respect. In dealing with the various specialist trades mention is made of the collapses that have occurred in reinforced concrete work when designs have been prepared by incompetent persons, and how essential it is for all those connected with the preparation of the drawings and the supervision of buildings of this class to be properly trained in the subject. The merits of reinforced concrete construction are put forward rather grudgingly, but nevertheless they are recognised, and the study of the subject is advocated.

The position of the architect is well defined and some good advice is offered on this matter. It is probably in experimental and scientific research that Germany has made more progress than any other country, and some examples are given of the comparative methods obtaining in this country, Germany and America which indicate the need for vast improvement on our part. With regard to the production of cement, it is stated that the value of the amount exported annually from Germany has increased prior to the War although the British article is superior.

It is true that the Germans do not hesitate to adopt shady methods if necessary to increase their sales, but generally speaking, their success is due to a better organisation and the technical training of their salesmen. The German technical schools are more numerous and generally better equipped, and their progress in technical education during recent years is quite extraordinary.

It should prove beneficial if the booklet is widely read as it is forcibly written, and the question of the recapture and expansion of British trade has been put forward in a very sensible manner.

The BRITISH STEEL PILING CO



FACTS SPEAK LOUDLY.

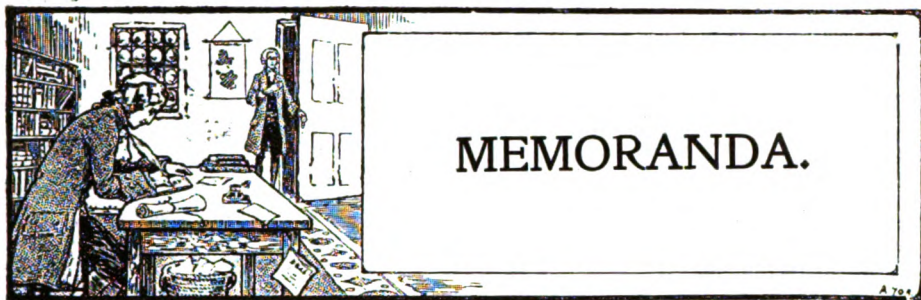
Photograph shows a cofferdam in the River Mersey made with "Universal Joist" Piling, 15" x 5" section (43 lbs. per sq. ft.) in 42½ ft. lengths, for construction of abutments of Transporter Bridge at Warrington. Excavation was made to within 2 ft. of toes of piles. Piling for one abutment only was supplied, and driven and withdrawn twice for two such cofferdams. It was withdrawn by means of one of our **Hydraulic Extractors**, and is now being used on another work.

NOTE LIGHT BRACING.

Weights of Piling on sale or hire :—

"SIMPLEX"	-	22 to 27 lbs. per sq. ft.
"UNIVERSAL JOIST"	-	from 43 lbs. "

DOCK HOUSE, Billiter Street, LONDON, E.C.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

City and Guilds of London Institute—Department of Technology.—The programme for the season 1915-16 has been issued, and contains the regulations for the registration, conduct, and inspection of classes, and examination of candidates in technological subjects. Detailed particulars are given as to the requirements for the different examinations, with dates of the examinations. Applications for the registration of classes in technology should be made not later than October 2nd at the offices of the Department of Technology, Exhibition Road.

The syllabus for the examination in structural engineering includes, in Grade 1, Loads on Structures; Bending Moments; Shearing Force; Moment of Resistance, Practical Details; Roof Trusses, Girders, Masonry Structures, Materials. The final examination takes in the above subjects in a more advanced form, and includes Bridge Construction, Wind Pressure, and Reinforced Concrete. For a copy of the programme and full details as to classes, etc., application should be made to the Department of Technology, City and Guilds of London Institute, Exhibition Road, S.W.

Reinforced Concrete Footbridge, Neasden.—The bridge is over the new fast lines of the Metropolitan Railway used for the through City trains. The structure was designed and executed by Mr. William Willox, the Company's Chief Engineer, and the whole structure is of plain but pleasing appearance.

The bridge and the staircases are entirely of reinforced concrete. The former is of box-shaped section, 6 ft. wide, 7 ft. 3 in. high, with a clear span of 35 ft.

The reinforcement is of mild steel bars to British standard specification for mild steel. They are bent, hooked at ends, formed into stirrups, shaped, and laced together at all junctions.

The concrete has been mixed most carefully; this was all the more essential as it is nowhere very thick. It is therefore necessary to describe its composition and manufacture rather minutely. Only clean, gritty, and sharp sand, free from organic, earthy and clayey matter or any other impurities, was used, particles of which had to pass a mesh of $\frac{1}{4}$ in. square, while 75 per cent. passed a mesh $\frac{1}{8}$ in. square. Sand not retained on a $\frac{1}{16}$ -in. square mesh was rejected. The sand, when necessary, was thoroughly washed. The aggregate consisted of screened gravel or broken ballast, clean, and varying in shape and size as much as possible. It passed through a $\frac{3}{4}$ -in. square mesh and was retained on one $\frac{1}{4}$ in. square. The preparation of the concrete was done as follows: The cement and sand were first mixed together by being turned over twice dry, this was then added to the ballast, and all the materials were well amalgamated by being turned over three times dry and then twice after the addition of a proper quantity of water from the rose of a water-can before being filled into the moulds. The mixture consisted of two bags of cement (each containing 204 lb.) to 7 cu. ft. of sand and 14 cu. ft. of aggregate. The gauge-box for the aggregate was 4 ft. by 4 ft. by $10\frac{1}{2}$ in., and that for the sand 3 ft. by 2 ft. 8 in. by $10\frac{1}{2}$ in. The amount of water used in gauging was sufficient to enable the materials to slide easily over each other and to be packed effectively into place. Under ordinary conditions of temperature this was obtained with about 23 gall. of water to each cu. yd. of dry material. The concrete, after being filled into the moulds, was thoroughly tamped around the bars by special tools and was thereby well rammed in position.

The concrete steps were cast in wrought deal moulds and were composed of two parts of sand of the specified quality, three of clean ballast, to pass a $1\frac{1}{2}$ -in. ring, and one of cement; in each step to its whole length, including the bearings, there was embedded two $\frac{1}{4}$ -in. by $\frac{1}{4}$ -in. bars with cross bars of hoop iron $\frac{3}{8}$ in. by 18 gauge at 1 ft. centres. The surfaces of treads were floated in the moulds to a thickness of $\frac{3}{4}$ in., with granite chippings to pass a $\frac{3}{8}$ -in. sieve, with cement gauged one to one. The face of the risers was similarly floated. The steps have a bearing of $4\frac{1}{2}$ in. at each end.

Concrete Plugs.—Concrete plugs have been used by the Warwickshire Coal Company at their colliery at Keresley, near Coventry, in order to stop the inrush of water, and as a result of the treatment in one shaft practically no water is being met with instead of 2,600 gal. per minute before applying this treatment. A concrete plug, 20 ft. thick, was placed in the bottom of each shaft which shut off completely the water and allowed of boring (twelve holes in each shaft) for the injection of cement from the bottom of the shafts.

Surface Imperfections.—Concrete-Cement Age states: "Imperfections in the exposed surfaces of concrete are usually due to one or more of ten well-known causes as follows:—

"(1) Variations in the nature of the cement, sand, or stone; (2) lack of uniformity in the amount of ingredients in each batch; (3) insufficient mixing in any or all batches of concrete; (4) lack of care in placing the concrete next to the moulds; (5) lack of proper protection in placing concrete; (6) efflorescence and discolouration of the surface; (7) unsightly construction joints; (8) imperfectly made forms; (9) dirt on the forms; (10) adhesion of forms."

ERRATUM.

In the article which appeared in our July number by Mr. Lance Martin, on p. 333, we regret the following errors have occurred: On p. 333 $r = b \times d$, in print a " b " has been put for the " d ." On p. 336, end of first paragraph "whether" should have been "where," and in example three on same page the multiplication $0.24 \times 21 = 5.04$, and not '285. Also $\sqrt{\quad}$ = on the second line from end should be deleted.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

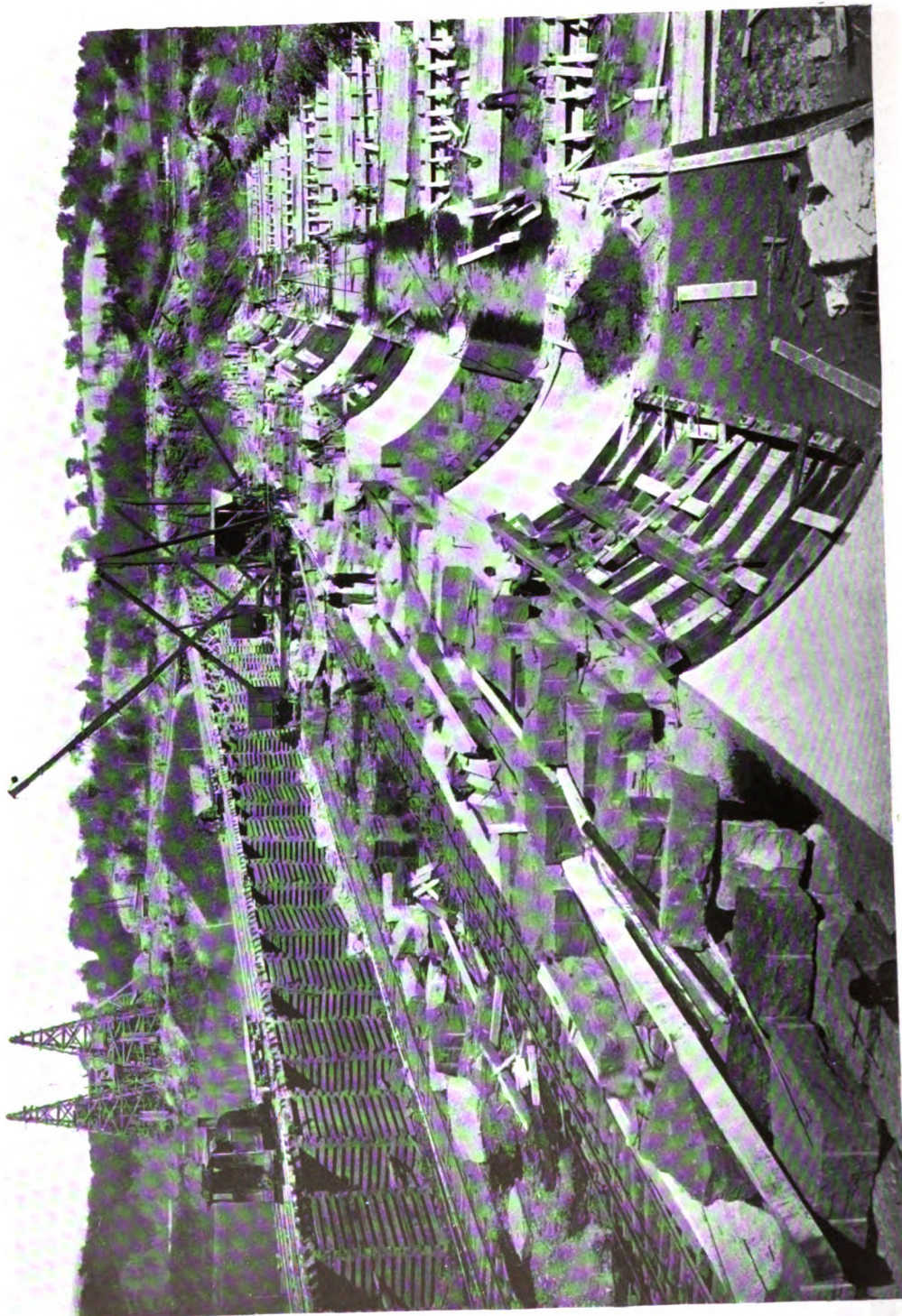
The **VICTORIA** is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



Dam at Vischer's Ferry, showing Cyclopean Masonry.
CONCRETE WORK ON THE NEW YORK STATE BARGE CANAL.
(See page 128.)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X. No. 10.

LONDON, OCTOBER, 1915.

EDITORIAL NOTES.

OIL-MIXED CONCRETE.

THE problem of rendering concrete waterproof is of such great practical importance that any effort to provide a solution is to be welcomed. There are certain defects inherent in any form of surface treatment, and preference is to be given to processes which render concrete impervious throughout. The addition of oil to the wet mixture has been recommended, especially in the United States, where this method has been frequently employed and investigated. That the addition of a mineral oil should lessen the permeability is only to be expected, but there is an obvious danger, namely, that a film of oil may surround the cement particles and not only retard the setting but also diminish the strength of the concrete by inhibiting those chemical and physical changes which constitute hardening.

We have previously published accounts of experiments in this direction, and we now publish an abstract of an important bulletin emanating from the U.S. Office of Public Roads, which contains a record of systematic investigations on this point. It appears from this that the effect of oil is to retard the setting, and consequently to diminish the crushing strength of mortar and concrete in short-period tests, but that the slow increase of strength due to subsequent hardening is scarcely interfered with by the addition of 10 per cent. of oil, so that after a year there is little difference of strength between the treated and untreated specimens. The elastic properties are little affected, whilst the permeability is very greatly reduced. These laboratory results are confirmed by practical experience. A few words of caution are needed, however. The oil must be chemically suitable, namely, a pure petroleum oil, free from animal or vegetable fats or asphaltic matter. Further, thorough mixing is required, the mixing time being twice as great as when oil is not used. Lastly, it must be remembered that no addition of waterproofing material will make up for bad proportioning or imperfect mixing. An ordinary concrete, scientifically proportioned to give a minimum of voids, has a very low permeability, which may be reduced almost to nothing by the proper addition of oil. On the other hand, a mixture which is highly porous will never become waterproof, however much oil may be added. It is likely that a simple process such as this, unencumbered by patent rights, will be largely employed in the treatment of tanks, retaining walls, basement floors,

and other structures where impermeability is essential, and it is to be hoped that records of practical experience in this country may soon be available.

PORTLAND CEMENT AND CEMENT CONCRETE IN SHIPBUILDING WORK.

We have from time to time given interesting particulars as to the use of Portland cement and Portland cement concrete for boats, barges, pontoons, ferries, and ships.

Italy was the pioneer in reinforced concrete barge building, and the barges and pontoons met with in Italy on the inland waterways are considerable in number and have proved efficient after stress and the ordinary wear and tear to which they are subjected. From the monetary point of view they are certainly cheaper in Italy than the steel barge or pontoon, and they have that particular advantage of not requiring constant painting and scraping, so necessary for a modern shell.

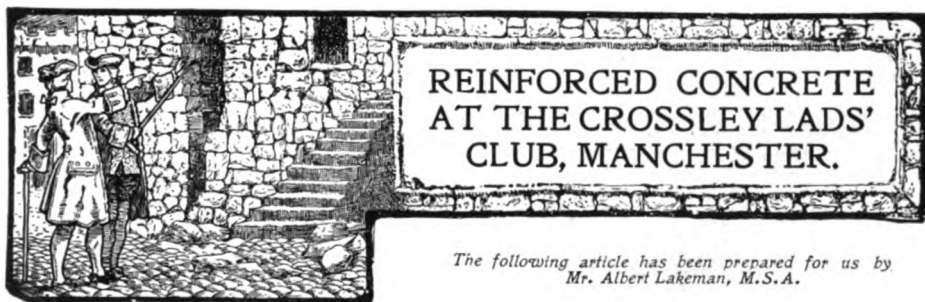
In another direction Portland cement has been found useful in shipbuilding, namely, as a protection of metal against corrosion; and, if the right kind of Portland cement is used, and if it is really reliably applied, nothing could be better.

Again, we meet with the use of Portland cement and cement concrete in ships—namely, for repair work; and it is a well-known practice of salvage companies and ship repairers to use quick-setting Portland cement or quick-setting cement concrete to fill up gaps made when a ship has been in collision or run against a rock.

Finally, we have the new requirements of modern warfare. It is well known that the hard woods, particularly teak, jarrah and karri, are frequently used for heavy scantlings as a backing to armour plate, but Portland cement concrete has been found to be a most useful substitute, having among other advantages that of being readily and quickly applied. By way of an example we would mention that the *Army and Navy Journal* reports that the backing of the armour plate of the United States battleship *Arizona*, now under construction at New York, is having a concrete-backed armour plate in lieu of teak, and in this instance the economy is emphasised, as both material and labour are so much cheaper.

Generally speaking, as far as this country is concerned, full use has not yet been made of Portland cement and cement concrete by our shipbuilding firms and ship repairers, and similarly not by our barge-builders and pontoon constructors.

We wish to call the special attention of these trades to the advantages of Portland cement and cement concrete at this stage, when economy, time, and labour are all matters of moment; and we trust that some of the more progressive firms concerned will not allow conservatism to override the claims of practicability and economy.



THIS building, which has been erected at Openshaw, Manchester, by Mabel Lady Crossley as a memorial to the late Sir William J. Crossley, Bart., is situated in Ashton Old Road, and covers a site measuring 69 ft. wide by 129 ft. 9 in. deep. The architect for the structure is Mr. John Broadbent, 71, Hale Road, Altrincham, and the accommodation provided covers both amusement and study. There are basement, ground and first floors, giving a total height of about 54 ft. from the lowest floor level to the ridge of the pitched roof. In the basement the front portion is devoted to five courts, and the back portion is occupied by a football room 66 ft. long and 40 ft. wide, and extending through a part of the height of the ground floor, with a running track in the form of a gallery on all four sides at a height of 9 ft. above the floor.

On the ground floor a large recreation hall is planned, this being about 80 ft. long and 66 ft. wide. The main staircase leads out of this hall at the back, and serves the main first floor in the front of the building and also the large gymnasium, which occurs at an intermediate level over the football room previously mentioned. In the front portion four class rooms and a large lecture hall are provided, the latter being 57 ft. long by 33 ft. wide, and having a raised platform at one end and a gallery at the other.

The structure is executed with a composite type of construction consisting of brick walls, steel roof trusses and reinforced concrete floors, columns, staircases and galleries, the whole of the latter being on the Mouchel-Hennebique system and executed in accordance with working drawings prepared for the architect by Messrs. L. G. Mouchel and Partners, of Westminster. The exterior of the building is illustrated in Figs. 1 and 2, and the first floor in Fig. 3, the latter giving a general idea of the lay-out, and it is interesting to note that all the partitions shown here are carried by the reinforced concrete flooring. The brick walls generally are designed to act as weight-carrying members, and there are only three reinforced concrete columns which go down to the soil. Of this number two occur in the front part of the building and extend through the ground floor and basement with an overall size of 12 in. square for the former and 15 in. square for the latter, the reinforcement in each case being four vertical rods well tied with links.

The safe load on the soil was taken as 2 tons per sq. ft., and the foundation slab is 6 ft. square with a minimum thickness of 6 in. at the outer edges and a maximum of 17 in. at the intersection with the column. The reinforcement in the slab consists of a layer of rods in both directions near the under surface and also at the centre of the thickness



with stirrups round the lower bars to provide anchorage and resistance to shear. The remaining column which bears on the soil extends through the basement only, and this is 9 in. square, with four lines of vertical reinforcement and a foundation slab 3 ft. 6 in. square, designed on the same principle as that mentioned above. A longitudinal section of the building is illustrated in Fig. 4, and it will be seen that other columns occur on the ground and first floor which are stopped at the ground floor level. A detail of one of these is shown in Fig. 9, this being 12 in. square with four vertical rods. It is carried by a brick wall which occurs in the basement, and a special slab of reinforced concrete is provided, this being 4 ft. long, 12 in. thick, and of the same width as the wall, which is 2 ft. 3 in. The slab is designed to distribute the load over



Fig. 2. Front Elevation.

REINFORCED CONCRETE AT THE CROSSLEY LADS' CLUB, MANCHESTER.

the requisite area of brickwork, and it is reinforced with rods in both directions and in both upper and lower surfaces with stirrups as indicated on the detail. The columns seen in the photographic view of the Recreation Hall, Fig. 7, are identical with the one described, and the architectural effect produced by them is quite pleasing. These columns are reduced to 10 in. square on the first floor. The floors are of two types, ordinary reinforced concrete slabs carried by secondary and main beams for the greater portion of the work, and special hollow reinforced concrete tubes used for the ground floor construction over the five courts in order to prevent the transmission of sound. Where the tubes are used the floor has a total thickness of 10 in., made up with a hollow tube 8 in. deep and 2 in. of concrete over same. These tubes, which are a patent type of Messrs. Mouchel and Partners, are convex on the top to give an arched effect which increases the strength, and they have sloping sides,

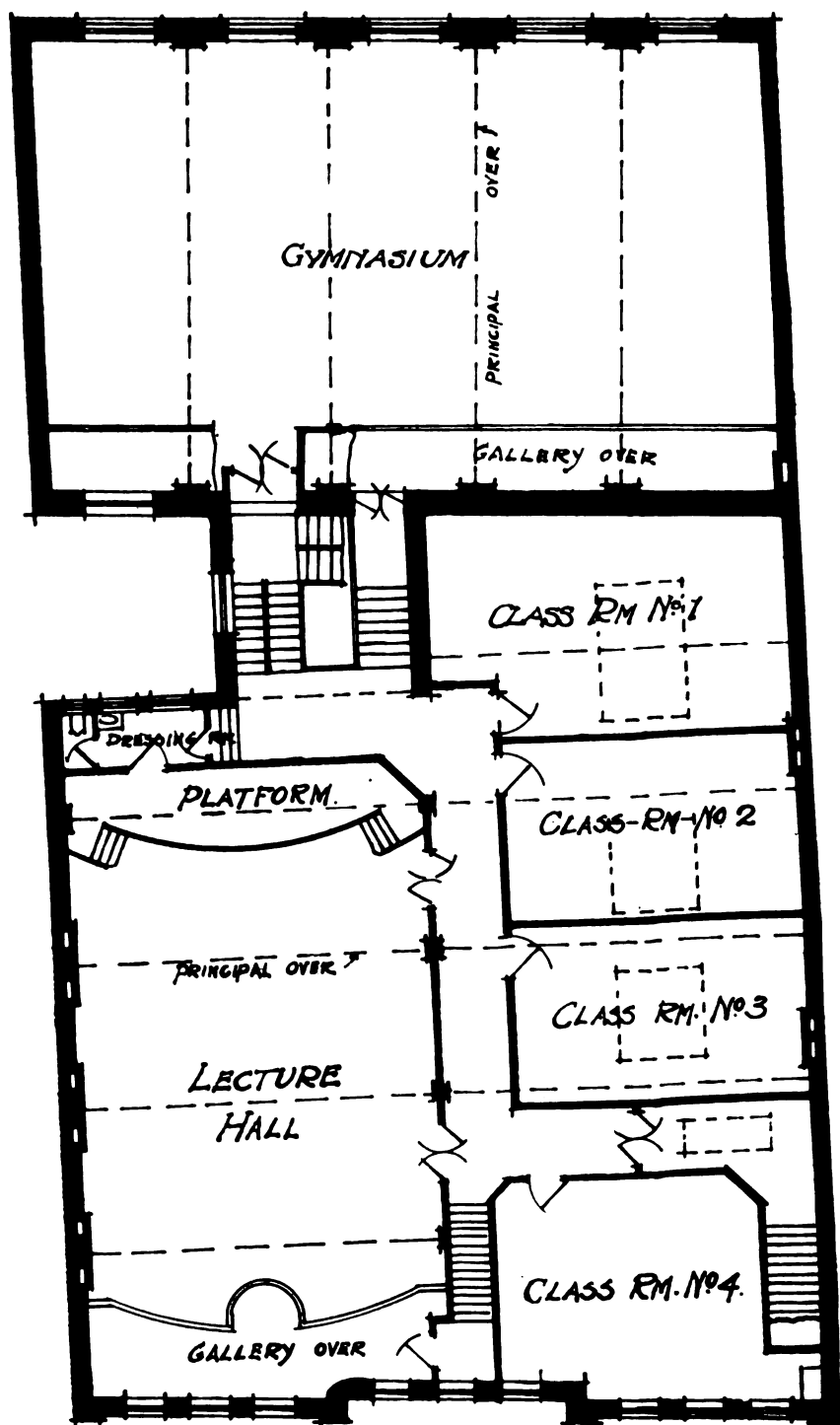


FIG. 3. First Floor Plan.
REINFORCED CONCRETE AT THE CROSSLEY LADS' CLUB, MANCHESTER.

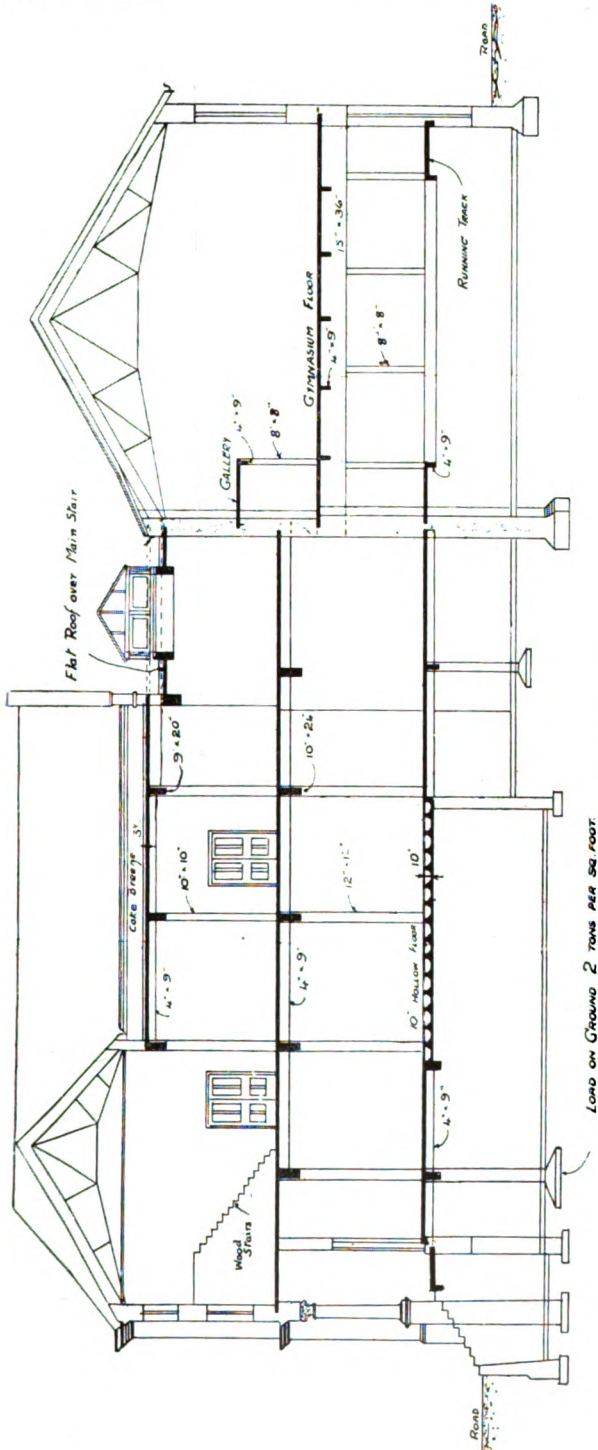


Fig. 4. Longitudinal Section.
REINFORCED CONCRETE AT THE CROSSLEY LADS' CLUB, MANCHESTER.

causing the bottom of the tube to be 1 in. narrower than the top, thus preventing any tendency to drop, and they are placed 4 in. apart, the spaces between being filled in with concrete reinforced in the lower surface to form small beams. Rods are also placed in the upper surface at right angles to the tubes, and the whole forms a light, sound-resisting floor, reasonable in cost, and quickly executed.

The detail of a main floor beam is illustrated in Fig. 6, there being several of this type at the first floor level. It has a span of about 32 ft., and supports the ends of eight secondary beams, and one end is carried by the main brick wall with a reinforced concrete template to distribute the load, and the other is taken by one of the columns mentioned previously as extending through the ground and first floors. The beam is 10 in. wide and 24 in. deep exclusive of the slab at the top, which is 3½ in. thick, and it is reinforced with four rods in the lower surface and two in the

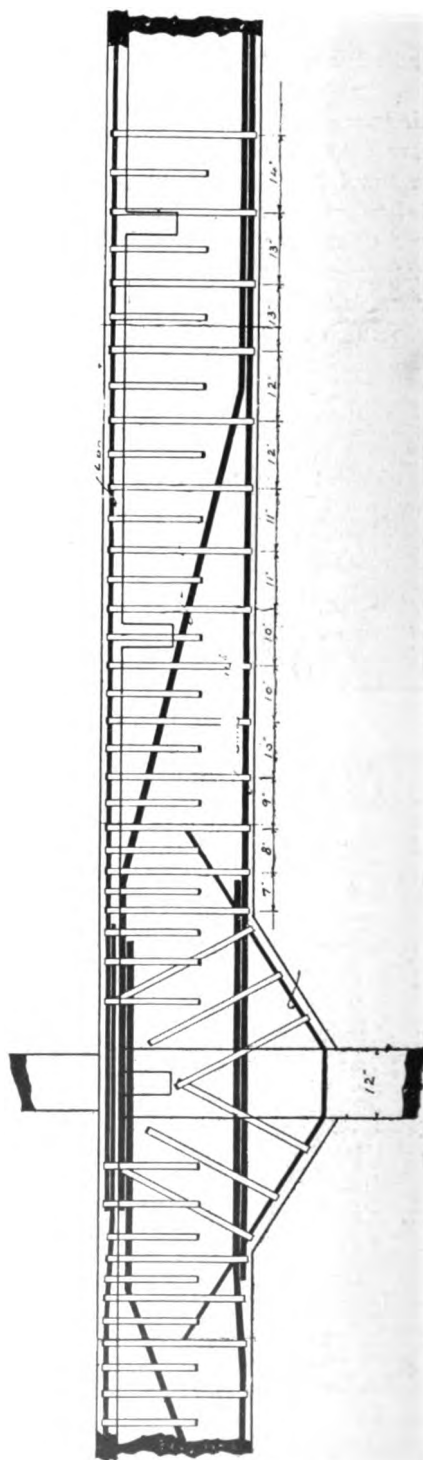
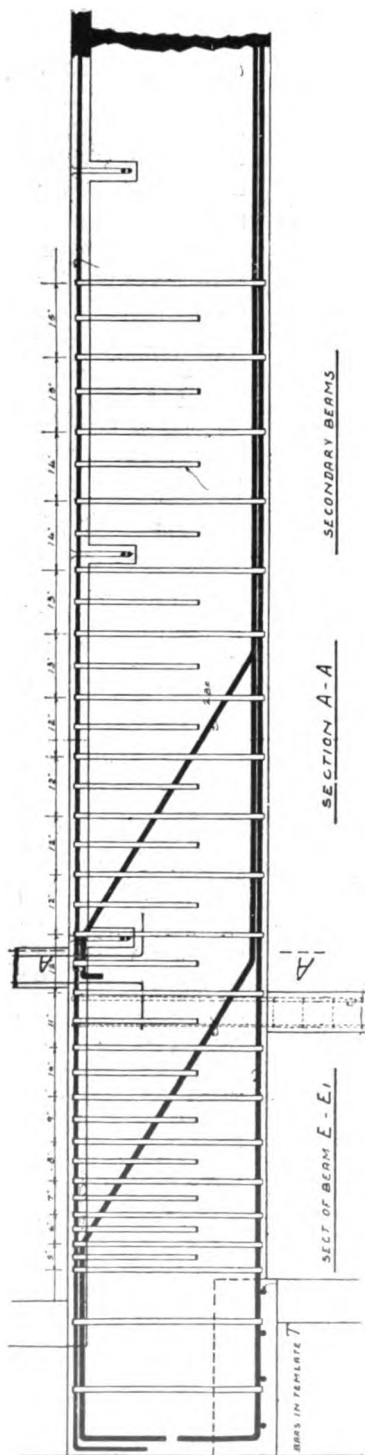


Fig. 6. Main Beam, First Floor Level.
REINFORCED CONCRETE AT THE CROSSLEY LADS' CLUB, MANCHESTER.



Fig. 7. Recreation Hall.

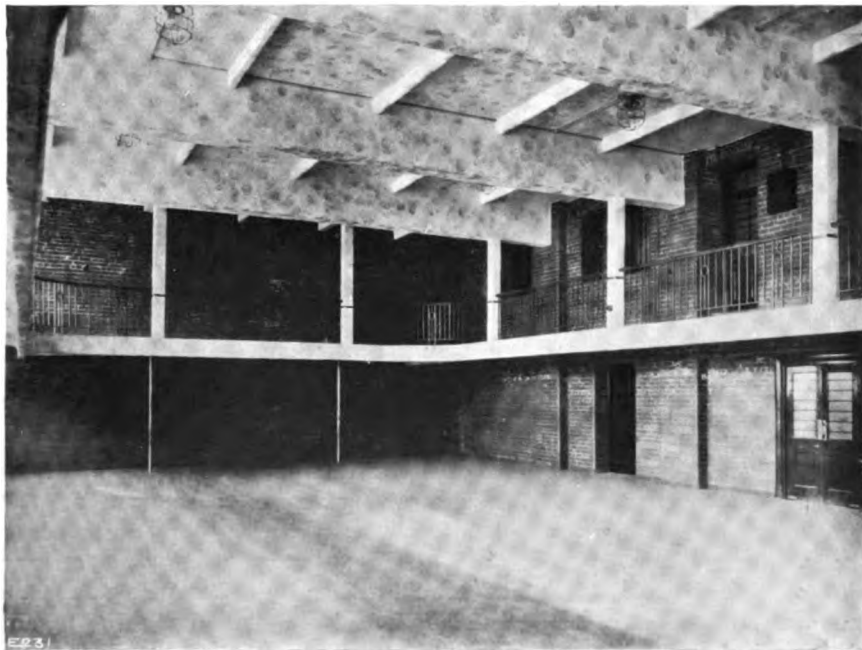


Fig. 8. Football Room.

REINFORCED CONCRETE AT THE CROSSLEY LADS' CLUB, MANCHESTER.

top, these being arranged as shown in the detail. The secondary beams, which have a span of 12 ft., are spaced about 6 ft. centres, and they are 4 in. wide and 9 in. deep below the floor slab, with two rods as reinforcement in the lower surface. The end of one main beam comes against a chimney breast, and a special beam 26 in. deep and 9 in. wide was provided to take the load and distribute it on the brickwork available in front of the flues, this beam being reinforced with four rods in both upper and lower surfaces. The floor loads generally were calculated at $1\frac{1}{2}$ cwts. per foot super. In order to keep the football room quite clear of obstructions, the gymnasium floor over is constructed with main beams which span right across the room, a distance of 40 ft. in the clear, and they are placed at about 12 ft. 6 in. centres.

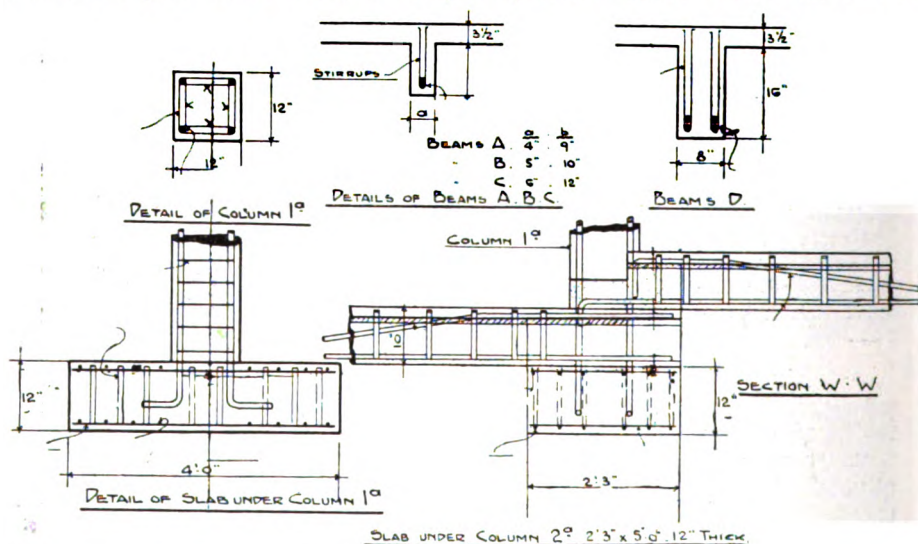


Fig. 9. Details of Columns and Beams.
REINFORCED CONCRETE AT THE CROSSLEY LADS' CLUB, MANCHESTER.

A detail of one beam is shown in Fig. 5, these being 15 in. wide and 36 in. deep below the $3\frac{1}{2}$ in. floor slab, with eight reinforcing rods in the lower surface and four in the upper. The effect of these beams can also be seen in the photographic view of the football room in Fig. 8, and it will be noticed that the photograph was taken after the room had been used, as the marks seen on the beams will testify. The secondary beams here are also 9 in. by 4 in., except in the cases where additional load was brought on to them by the running track, which is suspended entirely from the overhead construction with hangers. These hangers are 8 in. square, with four rods which are carried up and bent well over the rods in the upper surfaces of the beams, and at the foot they support a continuous 9 in. by 4 in. beam carried round on the outside of the running track. A gallery occurs on one side of the gymnasium, and a portion of the load from this is also brought down by columns on to the main beams over the football room, and thus a considerable load has to be taken by the latter. The gallery in the lecture hall on the first floor is also constructed in reinforced concrete, this being shaped on plan with a front 2 ft. 6 in. high

constructed as a beam and 8 in. square columns under the central curved portion, as seen in *Fig. 10*.

The reinforcement throughout consisted of spiral bond bars supplied by

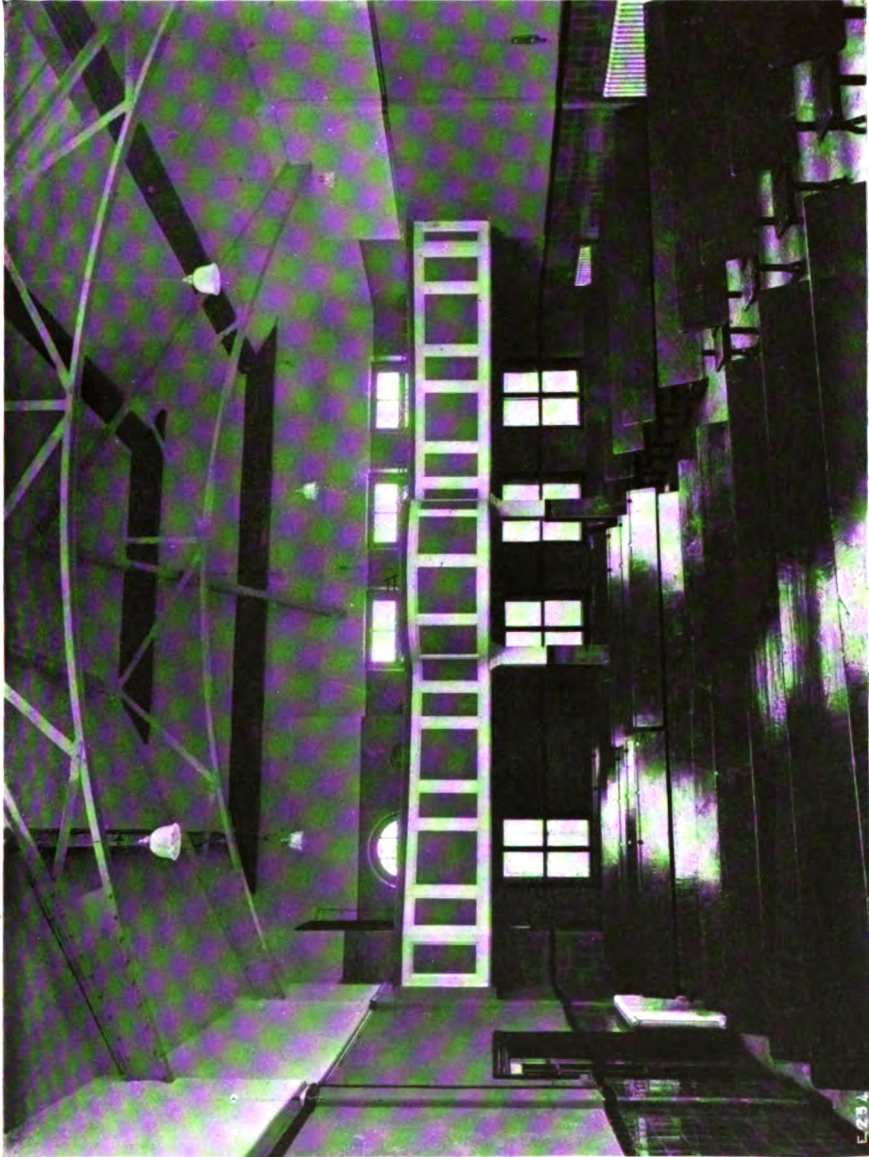
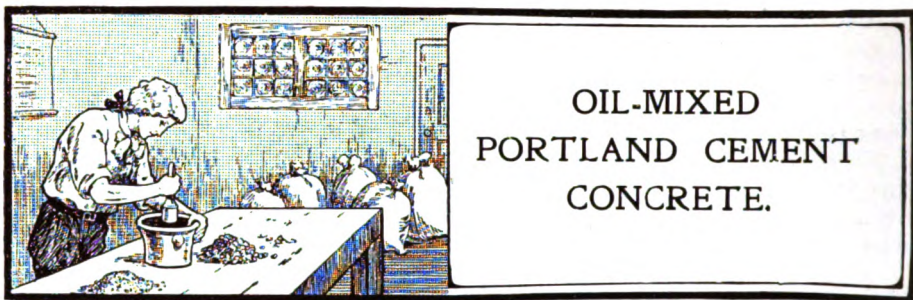


Fig. 10. Portion of Lecture Hall.
REINFORCED CONCRETE AT THE CROSSLEY LADS' CLUB, MANCHESTER.

the Spiral Bond Bar Co., of Caxton House, Westminster, it being found that economy and quick delivery could be obtained by the use of these bars. The general contractors were Messrs. William Thorpe and Son, of Manchester, and the whole of the ferro-concrete was carried out by the Yorkshire Hennebique Contracting Co., Ltd., of Viaduct Works, Leeds.



By LOGAN WALLER PAGE,

Director, Office of Public Roads, Washington, D.C.

The question of Oil-Mixed Portland Cement Concrete is one of considerable importance and interest. In Vol. VIII of this Journal, p. 619, we published some further experiments made by Mr. Logan Waller Page on this question. The Bulletin No. 46, from which we then printed, has now been revised and brought up to date by additional tables and information regarding new tests. We give below an abstract from the Revised Bulletin No. 230, from which we have also reproduced some diagrams and illustrations.—ED.

OIL-MIXED CONCRETE.

THESE tests have now extended over a period of considerably more than two years. Many valuable data have been obtained, through both laboratory and service tests, which demonstrate very definitely the worth of oil-mixed concrete in damp-proof and waterproof structures. Detailed results of these various tests are given in the Appendix.

With the view of determining what effect the addition of oil to cement mortar would have in retarding the action of alkali salts on the cement, a series of experiments was conducted which seemed to indicate that the action of the salt solution is materially retarded by the addition of 5 to 10 per cent. of oil to a 1 : 3 mixture. The illustration, page 493, shows a view of a series of briquettes subjected for one year to the action of a 10 per cent. solution of sodium sulphate. The briquettes in the upper row contained 10 per cent. of oil; those in the middle row 5 per cent. of oil; and those in the bottom row no oil.

SPECIFICATIONS FOR OIL TO BE USED IN OIL-CEMENT CONCRETE.

(Subject to revision.)

- (1) The oil shall be a fluid petroleum product and shall contain no admixture of fatty or vegetable oils.
- (2) It shall have a specific gravity not greater than 0.945 at a temperature of 25° C.
- (3) It shall show a flash point of not less than 150° C. by the closed-cup method.
- (4) When 240 cc. of the oil is heated in an Engler viscosimeter to 50° C., and maintained at that temperature for at least three minutes, the first 100 cc. which flows out shall show a specific viscosity of not less than 15 nor more than 30.
- (5) When 1 part of the oil is shaken up with 2 parts of hundredth normal caustic soda, there shall be no emulsification, and upon allowing the mixture to remain quiet the two components shall rapidly separate in distinct layers.

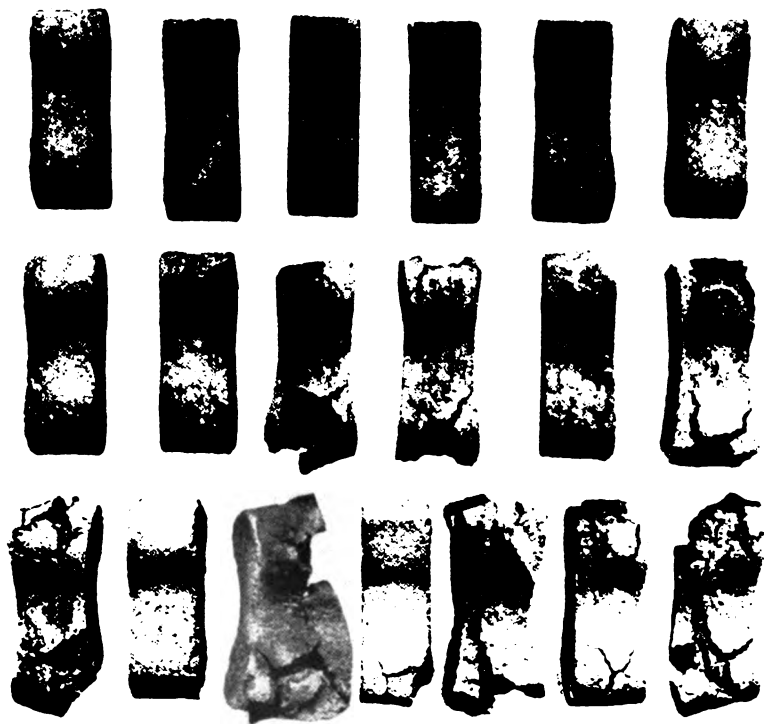
The general purpose of the above clauses is as follows:—

Clause 1 eliminates compounded products in which the presence of saponifiable oils would break down the strength of the cement. Clause 5 has a similar purpose in eliminating certain straight petroleum residuals which readily emulsify with alkali, and seriously impair the strength of the mortar to which they are added. Clauses 2, 3, and 4 combine to prevent the use of certain asphaltic oils which prove detrimental to the strength of the concrete, and Clause 4, in particular, prescribes an oil of such viscosity as to be readily miscible with the mortar, while still possessing sufficient body to render the structure damp-proof.

METHOD OF MAKING.

For most purposes where damp-proofing is required 5 per cent. of oil based on the weight of cement in the mixture is all that is necessary. A bag of cement weighs 94 lb., and consequently, for each bag of cement used in the mixture, 4·7 lb. or about 2½ quarts of oil are required.

Let it be supposed that a batch of concrete requiring two bags of cement is to be mixed in the proportions of 1 part of cement to 2 parts of sand to 4 parts of broken stone or gravel, together with 5 per cent. of oil. Four cu. ft. of sand are first measured out in a bottomless box 12 in. deep and 2 ft. on each side. On top of the sand is



1 BRIQUETTES SHOWING REPELLENT ACTION OF OIL-CEMENT CONCRETE ON ALKALI WATER.
Top row contained 10 per cent. of semiasphaltic oil; middle row contained 5 per cent. of semiasphaltic oil; bottom row contained no oil. Briquettes were immersed one year in a 10 per cent. solution of sodium sulphate.

spread the cement, and these materials are mixed together until they appear to be of uniform colour. Water is then added to the mixture and the mass again mixed to a mortar of mushy consistency. Five quarts of oil are then measured out and added to the mortar, and the mass again turned until there is no trace of oil visible on the surface of the mortar. Particular care should be taken to continue the mixing until the oil is thoroughly incorporated in the mixture. Experience has shown that to insure the very best results the length of time of mixing should be practically double that required when oil is not used. The oil-mixed mortar is then combined with the stone or gravel previously moistened and the mass is again turned until all of the stone is

thoroughly coated with the mortar and the mass is uniformly mixed throughout. Should only oil-mixed mortar be desired, the process is similar to that above described, except that no stone is added.

In a machine mixer the cement, sand, and water are first mixed to a mortar, when alternate batches of oil and stone are added until the required quantity of oil is mixed, and then the remainder of the stone is added and mixed. When a batch mixer is used, the exact method of procedure should be determined by experiment, owing to the fact that different makes of mixers require slightly different handling to insure best results. A continuous mixer should not be used in oil-cement-concrete work, as with this type the time of mixing cannot readily be increased to the extent necessary to insure a uniform distribution of the oil.

MATERIALS REQUIRED FOR 1 CU. YD.

The following table gives the proportions by parts and amounts required of cement, sand, stone, and oil to make a cubic yard of oil-mixed mortar and concrete:

TABLE 2.—QUANTITIES OF MATERIALS REQUIRED FOR 1 CUBIC YARD OF OIL-MIXED MORTAR AND CONCRETE.

Proportions by parts.				Cement (barrels ¹).	Sand (cubic yards).	Stone or gravel (cubic yards).	Oil (gallons ²).
Cement.	Sand.	Stone or Gravel.	Oil (per cent.).				
1	—	—	3	8.31	—	—	12.1
1	2	—	5	3.32	0.93	—	8.06
1	3	—	5 10	2.48	1.05	—	6.02 12.04
1	4	—	5 10	1.98	1.11	—	4.8 9.61
1	2	4	5	1.57	0.44	0.88	3.81
1	2½	5	5 10	1.30	0.46	0.92	3.15 6.3
1	3	6	5 10	1.11	0.47	0.94	2.69 5.38

¹ One barrel of cement equals 4 bags.

² Oil weighs about 7½ lb. per gallon.

USES.

Irrigation Ditches.—The results of laboratory tests, which indicate that the presence of oil tends to retard very materially the action of alkalis on concrete, suggest that another field for the use of oil-cement concrete may be found in the construction of linings for irrigating canals and ditches. Many of these canals are in localities where the soil is strongly impregnated with alkali salts and where the water carried contains alkali in solution. The destructive action of alkali is undoubtedly due to the crystallisation of the salts within the mass of the concrete, or the formation by chemical action of compounds of greater volume than the original salts, or to a combination of both of these actions.

As the admixture of oil will retard the absorption of water into the concrete, it should materially lengthen the life of the lining. In the mixing of concrete for this purpose it is, of course, necessary to avoid the use of either water or sand containing alkali.

Concrete Base for Roadways.—The use of this material should also prove of value for damp-proofing the concrete base of roads against the action of ground water, which if allowed to pass through will tend to disintegrate the road surface. Such action as this is particularly noticeable with road surfaces such as asphalt, bituminous concrete, etc. Assuming the usual proportions for the concrete base, etc., 10 per cent. of oil should prove sufficient for this purpose.

APPENDIX.

PHYSICAL TESTS OF OIL-MIXED PORTLAND CEMENT CONCRETE.

In order to investigate the physical properties of oil-mixed Portland cement concrete, the following physical tests were conducted in the testing laboratory of the Office of Public Roads: (1) Tensile strength, (2) crushing strength, (3) time of setting, (4) toughness or resistance to impact, (5) stiffness or modulus of elasticity, (6) absorption, (7) permeability, and (8) bond tests.

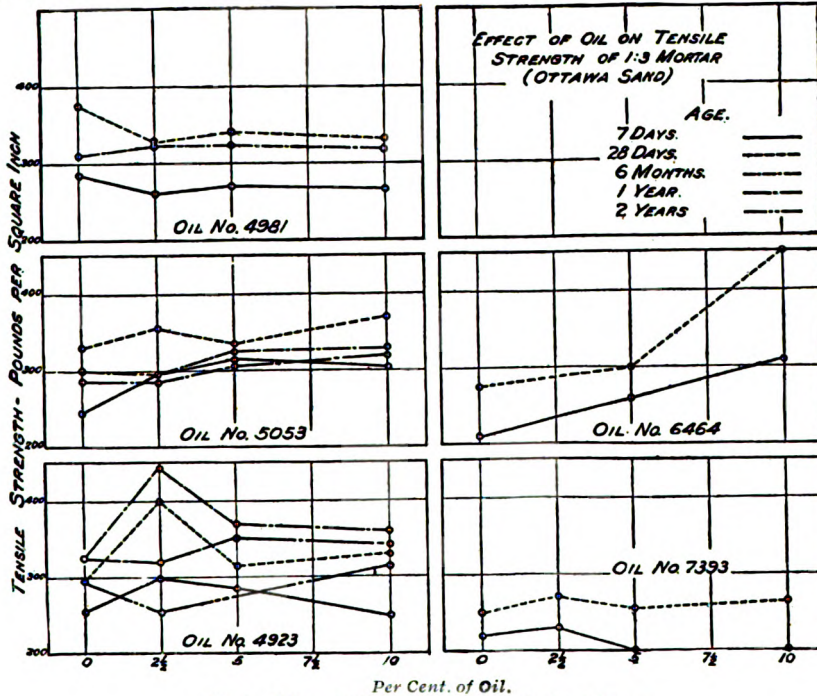


Fig. 1. Effect of Oil on Tensile Strength of 1:3 Mortar.

The materials used consisted of Portland cement, river sand, crusher-run gneiss, river gravel, and various kinds of petroleum residuum oils.

Various types of oils were used, and these are described in the following table:

ANALYSIS OF OILS USED IN OIL-CEMENT-CONCRETE MIXTURES.

	Sample No.									
	4145	4146	4147	4149	4170	4923	4981	5053	6464	7393
Type	(1)	(1)	(1)	(1)	(2)	(1)	(1)	(1)	(1)	(1)
Character	(2)	(2)	(2)	(2)	(4)	(2)	(2)	(2)	(2)	(2)
Specific gravity at 25°-25° C. ..	0.924	0.910	0.926	0.923	—	0.945	0.893	0.924	0.904	0.949
Per cent. of loss at 163° C., 5 hours (20 grams)	6.86	12.56	7.98	7.02	18.38	1.35	27.17	3.70	0.10	19.38
Character of residue	(3)	(3)	(3)	(3)	(5)	(3)	(7)	(3)	(2)	(4)
Per cent. of bitumen soluble in CS ₂ air temperature	99.99	99.99	99.93	99.95	99.81	99.96	99.95	99.90	99.90	99.95
Per cent. of organic matter insoluble	0.01	0.01	0.07	0.05	0.13	0.04	0.02	0.07	0.10	0.05
Per cent. of inorganic matter insoluble	0.00	0.00	0.00	0.00	0.06	0.00	0.03	0.03	0.00	0.00
Total (per cent.)	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
Per cent. of total bitumen insoluble in 86° B. paraffin naphtha ..	2.23	6.82	10.16	2.24	16.87	3.46	1.00	4.12	1.10	2.45
Fixed carbon	2.41	3.36	5.11	1.98	—	4.18	1.77	2.82	1.17	2.87
Specific viscosity, Engler, 50° C. ..	14.20	6.40	18.20	20.20	—	65.10	2.50	17.40	13.50	12.20

¹ Fluid residual oil.

² Cut-back oil asphalt.

³ Fluid, greasy.

⁴ Fluid, sticky.

⁵ Fluid, slightly sticky

⁶ Semisolid, sticky.

⁷ Slightly granular, more viscous than original.

⁸ Fluid, granular in appearance.

⁹ Very viscous, greasy fluid, rather lumpy; surface waxy.

TENSILE STRENGTH.

TENSILE STRENGTH (1 : 3 MORTAR, OTTAWA SAND).

Age in days.	Per cent. of Oil.	Laboratory No. of Oil.				
		4923	4981	5053	6464	7393
7	0	256	283	244	210	221
	2½	299	259	297	—	230
	5	287	268	313	258	200
	10	252	264	304	308	195
28	0	296	376	331	275	250
	2½	400	327	353	—	270
	5	316	341	334	298	253
	10	331	329	371	403	260
180	0	326	312	302	—	—
	2½	449	323	298	—	—
	5	372	321	325	—	—
	10	360	319	330	—	—
360	0	326	—	288	—	—
	2½	320	—	286	—	—
	5	353	—	309	—	—
	10	343	—	320	—	—
720	0	293	—	—	—	—
	2½	257	—	—	—	—
	5	—	—	—	—	—
	10	315	—	—	—	—

The foregoing results are plotted on *Fig. 1*. It will be noticed that, in general, the specimens containing oil have a higher tensile strength than those without oil.

CRUSHING STRENGTH.

Specimens of mortar and concrete containing different percentages of various kinds of oils were moulded 6 in. in diameter and 6 in. high. They were bedded in plaster of Paris and blotting paper, and crushed at a speed of 0.152 in. per minute. The results are plotted on *Figs. 2* and *3*. The crushing strength is decreased by the addition of oil, but the decrease is not serious when the amount of oil does not exceed 10 per cent. Also oil-mixed cement concrete gains in strength with time in about the same ratio as untreated concrete. This fact indicates that the addition of oil to the mixture in small amounts has no disintegrating effect on the cement. This statement holds true for periods up to and including two years.

The time of setting is delayed with the addition of oil. Five per cent. of oil delays the initial set by 50 per cent. and the final set by 47 per cent.

TOUGHNESS OR RESISTANCE TO IMPACT.

The toughness or resistance to impact was tested on the Page impact machine under the blows of a 10-kilogram hammer falling on a 5-kilogram plunger from successively increasing heights of 1 centimetre. The height of the blow causing failure corresponds to the number of blows. The end of the plunger in contact with the specimen is spherical in shape, with a radius of 3 centimetres. Specimens 6 in. in diameter and 6 in. high were tested after first bedding them in plaster of Paris before mounting on the anvil of the machine. The following results show that the toughness of concrete is very little influenced by the addition of a small amount of oil to the mixture :

NUMBER OF BLOWS REQUIRED TO PRODUCE FAILURE.

Per cent. of Oil No. 4923	1 : 3 : 5 Concrete.		1 : 3 : 6 Concrete.	
	Air-cured	Water-cured.	Air-cured.	Water-cured.
0	18	15	15	21
2½	15	33	14	18
5	23	20	15	20
10	13	20	12	21
15	14	20	13	16

A view of the impact machine with a specimen under test is shown in *Fig. 6*.

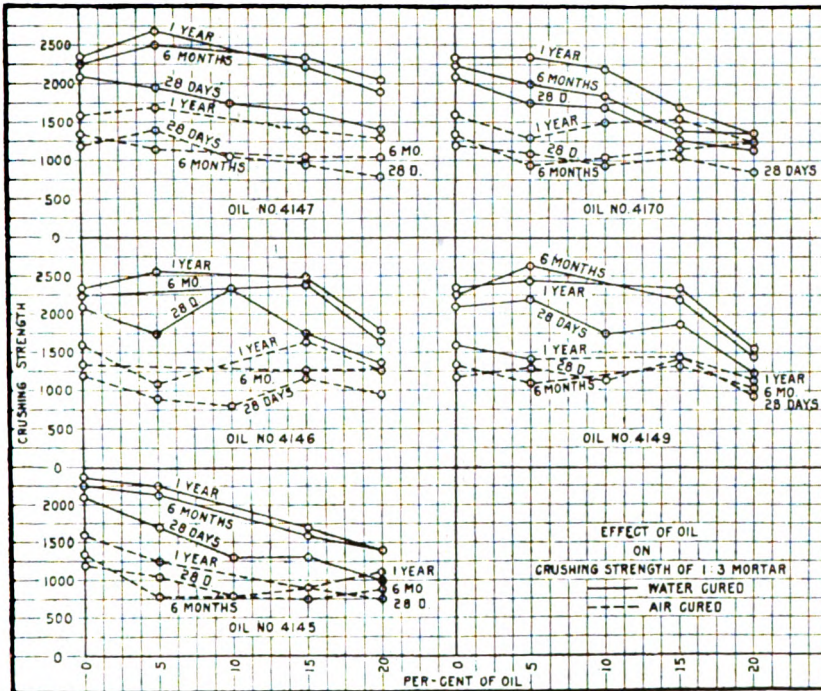


Fig. 2. Effect of Oil on Crushing Strength of 1:3 Mortar.

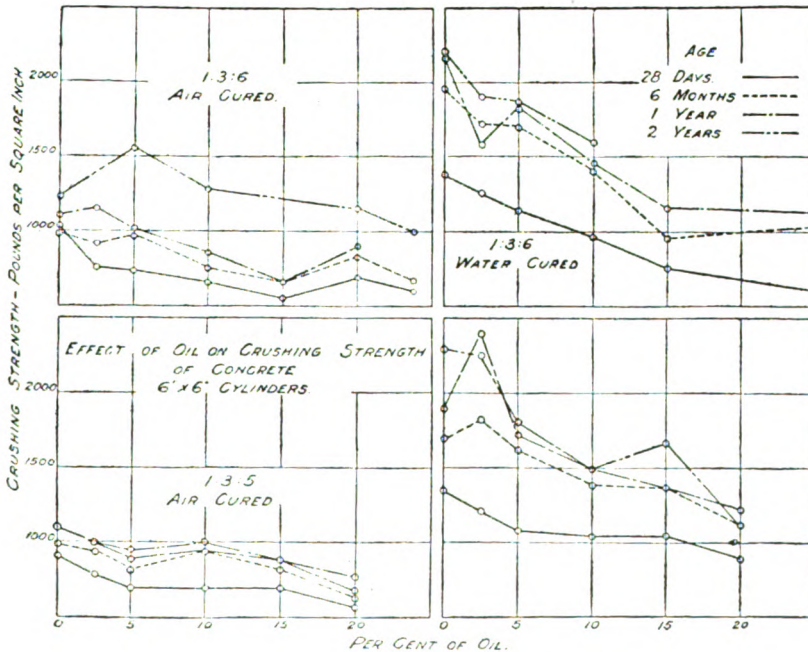


Fig. 3. Effect of Oil on Crushing Strength of Concrete.

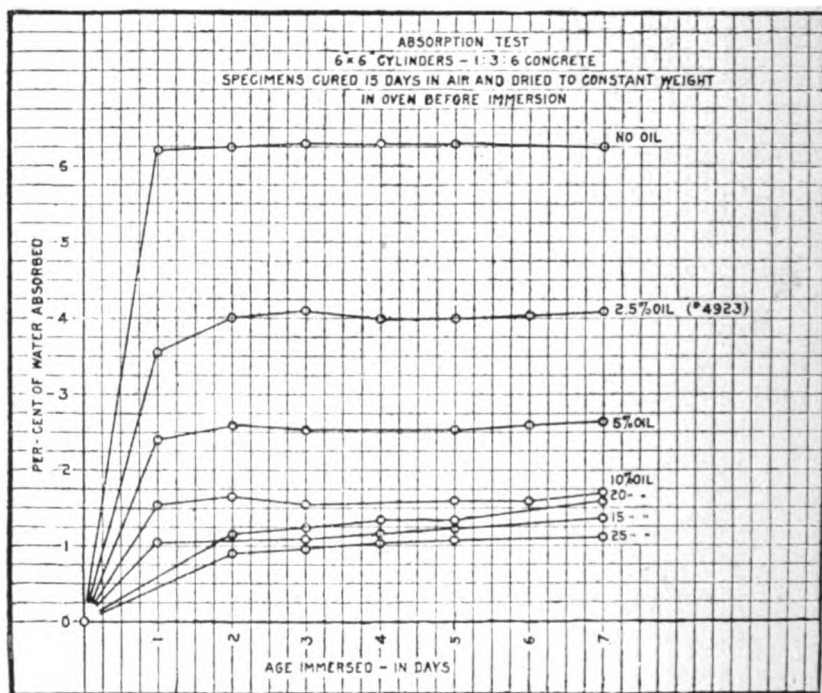


Fig. 4. Absorption Test.

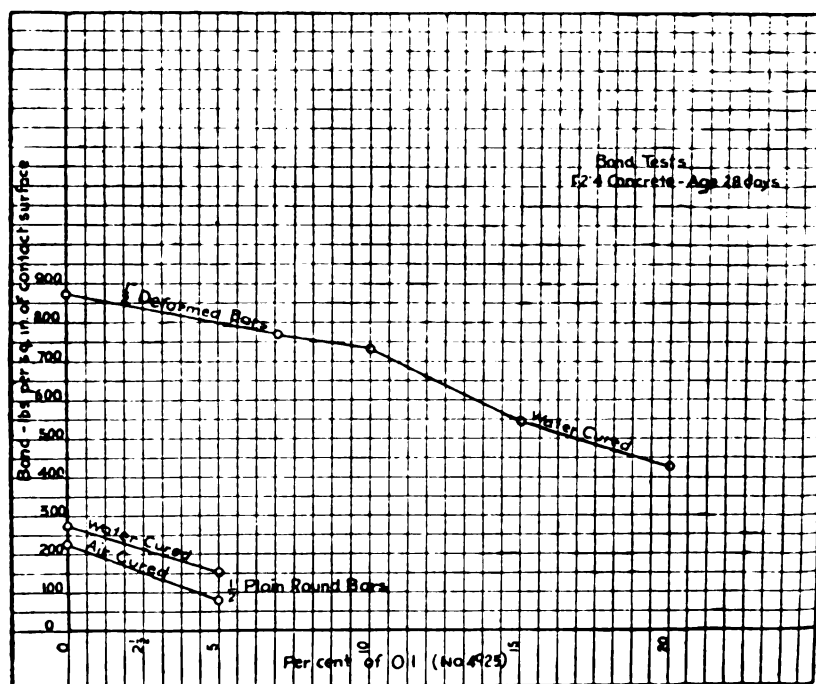


Fig. 5. Bond Tests.

STIFFNESS OR MODULUS OF ELASTICITY.

For testing the effect of oil on the stiffness of concrete, specimens 8 in. in diameter and 16 in. high were made. The deformations under various loads were measured with a double micrometer-screw compressometer. Loads were applied in

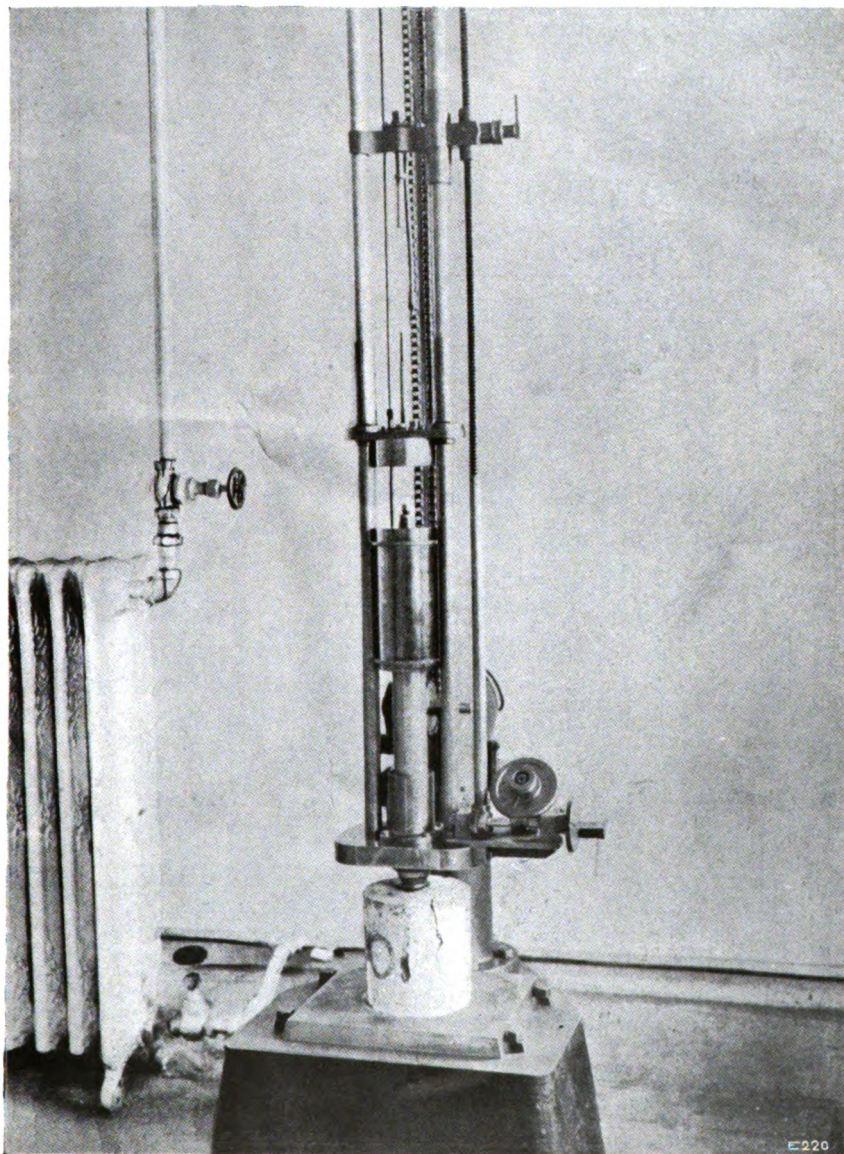


Fig. 6. Impact Test on Oil-Mixed Concrete.

2,500 and 5,000 lb. increments and were released to 500 lb. after each increment of 5,000 lb., and deformation readings were taken for permanent set. In all cases the initial modulus of elasticity was obtained from the slope of the stress-strain curve at its origin. A view of a specimen mounted in the testing machine with compressometer attached is shown in Fig. 7.

The results show that oil has little effect on the stiffness of concrete. The

c

increased value of the modulus of elasticity of the water-cured over the air-cured specimens is as marked in the oil-mixed as in the plain specimens. Tests at one year, although not here recorded, show that oil-mixed concrete gains as much in stiffness with age as the plain concrete does.

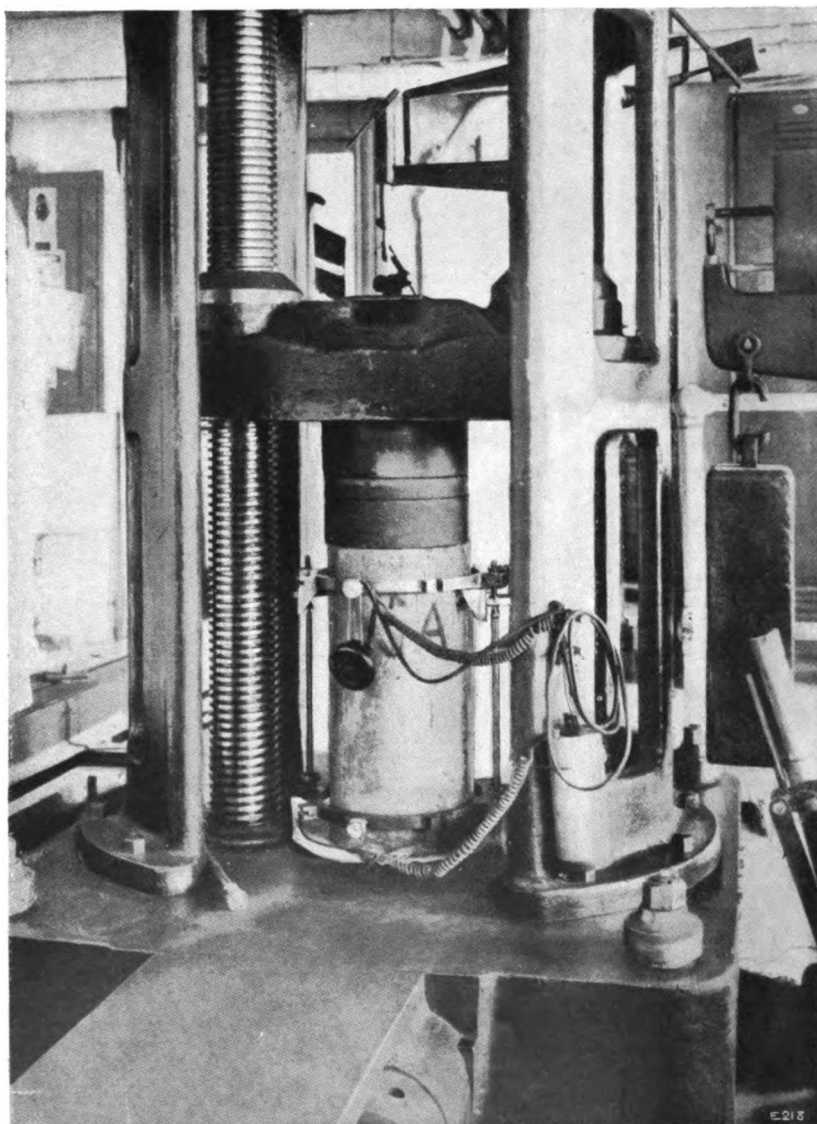


Fig. 7. Testing the Strength and Elasticity of Oil-Mixed Concrete.

ABSORPTION.

The resistance of concrete to the penetration of moisture is measured by its absorptive qualities. To test the absorption of oil-mixed concrete compared with plain concrete, cylindrical specimens 6 in. in diameter and 6 in. high were dried to constant weight in an oven, after being cured for 15 days in air. They were then immersed in water and weighed from time to time. The results of these tests are plotted on Fig. 4. It will be seen that the oil greatly decreases the percentage of

absorption; the cylinder containing 10 per cent. of oil absorbed 1.7 per cent. of water, based on the dry weight, while the cylinder containing no oil absorbed 6.25 per cent.

PERMEABILITY.

To investigate permeability, specimens 3 in. in thickness and 6 in. in diameter were encased with a surrounding ring of 1:1 mortar. Before testing, the top and bottom surfaces were chipped off in order to eliminate the waterproofing effect of the rich surface layers. Plain 1:3 mortar at the age of 28 days under 30 lb. pressure became damp after half an hour. Under 40 lb. pressure the leakage amounted to 146 cu. cm. after 24 hours' application. Specimens containing 5 and 10 per cent. of oil No. 4923 remained perfectly tight under 40 lb. pressure.

The results of all permeability tests seem to indicate that the resistance to water pressure is dependent more on the care used in proportioning and mixing the specimens than upon the addition of any extraneous waterproofing materials.

BOND TESTS.

To determine the adhesion of oil-mixed concrete to steel reinforcement, bond tests were made on specimens mixed in the proportions of 1:2:4 and containing various percentages of oil. Rods 12 in. long were embedded in the centre of cylinders 8 in. in diameter and 8 in. long. The test consisted in pushing the rods through the concrete, and the point of failure was taken at the drop of the scale beam.

Two kinds of bars were used—plain and deformed. All specimens were tested at 28 days, and the results are plotted on Fig. 5. The bond strength is decreased, and the decrease depends directly on the quantity of oil in the mixture. It is evident that the bond between plain bars and concrete is so seriously affected by the mixture of oil that it would be inadvisable to use such a combination. The bond of deformed bars is not so seriously affected, but is somewhat decreased by the oil admixture.

SUMMARY OF CONCLUSIONS.

The following conclusions as to the effect of the oils used in cement and concrete may be drawn from the foregoing investigations:

(1) The *tensile strength* of 1:3 oil-mixed mortar is very little different from that of plain mortar, and shows a substantial gain in strength at 28 days and at 6 months over that at 7 days.

(2) The *times of initial and final set* are delayed by the addition of oil; 5 per cent. of oil increases the time of initial set by 50 per cent. and the time of final set by 47 per cent.

(3) The *crushing strength* of mortar and concrete is decreased by the addition of oil to the mix. Concrete with 10 per cent. of oil has 75 per cent. of the strength of plain concrete at 28 days. At the age of 1 year the crushing strength of 1:3 mortar suffers but little with the addition of oil in amounts up to 10 per cent.

(4) The *toughness* or resistance to impact is but slightly affected by the addition of oil in amounts up to about 10 per cent.

(5) The *stiffness* of oil-mixed concrete appears to be but little different from that of plain concrete.

(6) *Elasticity*.—Results of tests for permanent deformation indicate that no definite law is followed by oil-mixed concrete.

(7) *Absorption*.—Oil-mixed mortar and concrete containing 10 per cent. of oil have very little absorption and under low pressures both are waterproof.

(8) *Permeability*.—While the laboratory tests to determine the waterproofing qualities of oil-cement concrete have not given uniform results, those made on oil-mixed cement mortar containing 10 per cent. of oil have shown that such mortar is practically waterproof under pressures as high as 40 lb. per sq. in. All the tests, whether in the laboratory or in construction work, indicate that oil-mixed mortar is very effective as a waterproofing agent under low pressures, when plastered on either side of a porous concrete or masonry wall.

(9) The *bond tests* show the inadvisability of using plain bar reinforcement with oil-concrete mixtures. The bond of deformed bars is not seriously weakened by the addition of oil in amounts up to 10 per cent.

NOTE.—The addition of any waterproofing or dampproofing agent to cement, mortar or concrete is valueless unless extreme care is exercised in proportioning, in mixing, and in placing the concrete.



The first 2 parts of these Regulations appeared in the September number.—ED.

PART III.

BEAMS.

Beam Defined.—55. The term “beam” shall include any beam, girder, lintel, bressummer or cantilever, or any other similar member carrying transverse loads.

Diameter of Bars.—56. The least diameter or thickness of the longitudinal bars in beams shall not be less than one-quarter of an inch.

57. All other reinforcements in beams shall be at least one-eighth of an inch in diameter or thickness.

Space between Bars.—58. There shall be a distance of at least one inch horizontally and one half-inch vertically between the bars in beams except at joints or at points where the bars are in direct contact and transverse to one another.

59. The distance between the bars of the tensile reinforcement in a beam shall not be greater than six inches.

Wiring.—60. Wiring used in beams for the purpose of holding bars in position shall not be regarded as reinforcement.

Compressive reinforcement.

61. In cases where the compressive resistance of the concrete is taken into account the stress in the compressive reinforcement may be taken at an amount not exceeding m times the stress in the concrete at the same distance from the neutral axis on condition that—

(a) The compressive reinforcement is anchored by bars extending at least through a depth equal to the arm of the resistance moment.

(b) The anchors are spaced not further apart (centre to centre) than a distance equal to the arm of the resistance moment, and not further apart (centre to centre) than 16 times the least diameter of the anchored bar. (*Also see regulations 89 and 90.*)

62. In cases where the compressive resistance of the concrete is not taken into account the stress in the compressive reinforcement may be taken at 16,000 pounds per square inch on condition that—

(a) Sufficient steel is provided to take up the whole of the compression.

(b) The compressive reinforcement is anchored laterally and vertically by anchors not further apart (centre to centre) than 6 inches and not further apart (centre to centre) than 8 times the diameter of the anchored bar.

(c) The anchors shall be passed round or hooked over both the compressive and the tensile reinforcement.

Hooped Core.—63. In cases where the concrete in compression is hooped at the ends of a beam, the stress within the hooped core, for the distance of one-quarter the span of the beam at each end, may be taken at the same value as for a core in a pillar similarly hooped. In this case the compressive stress in the concrete outside the hooping shall not be taken into account in estimating the compressive resistance.

Shear or web reinforcement.

Shear.—64. The vertical shear taken by the concrete only shall be calculated on the compressed area of the web or on the web area for a depth equal to the arm of the

resistance moment of the beam. The intensity of the shearing stress shall not be greater than the values given in regulation 42.

65. Where the vertical shear is taken by the concrete only, in accordance with regulation 64, the ends of 50 per cent. of the bars of the tensile reinforcement shall be inclined across the neutral plane of the beam, and shall be carried through a depth equal to the arm of the resistance moment, or the whole of the bars shall be carried through to the ends of the beam. (See regulations 87 (c) and 88 (c).)

66. If the shearing stress at any cross section, calculated on the concrete alone, is in excess of the permissible shearing stress, the whole shear shall be provided for by the tensile resistance of the shear or web reinforcement acting in conjunction with the compressive stresses in the web, but in no case shall the ratio $\frac{S}{b.d}$ exceed three times

the shearing stress given in regulation 42 (a), where

b_r = mean *breadth* of the *rib* of a tee beam or the *breadth* of a *rectangular* beam;

d = effective *depth* of the beam;

S = total vertical *shearing* force at any cross section.

(Also see regulation 84.)

67. Shear or web reinforcement shall—

(a) Be spaced according to the distribution and intensity of the shearing stresses, but the distance from centre to centre of the shear or web members at any part of the beam shall not exceed a length equal to the arm of the resistance moment.

(b) At least extend from the centre of the tensile reinforcement to the centre of pressure in the concrete under compression.

(c) Be passed under or round the tensile reinforcement or be otherwise secured thereto.

(d) Be hooked at both ends, in the same manner as for tensile reinforcement, or equally effectively anchored.

68. Tensile reinforcement which is inclined across the neutral plane of a beam, and which is carried through a depth equal to the arm of the resistance moment, may be taken as shear or web reinforcement.

69. The shear or web reinforcement may be regarded as anchors under the regulations 61 and 62, provided it complies therewith.

Splays and brackets at ends of beams and bracketed work.

Splays.—70. Where the end of a beam is splayed for the purpose of increasing the resistance moment, such splayed work shall not be calculated at a greater angle than 30 degrees from the horizontal.

Brackets.—71. In cases where bracketed work in reinforced concrete is provided in the structural design to carry loads, and to act as a cantilever, it shall be reinforced sufficiently to provide for the stresses due to such loads.

SLABS.

Effective Depth.—*72. The effective depth of slabs shall be measured from the compressed edge of the constructional concrete to the centre of gravity of the tensile reinforcement. In the case of floor slabs the effective depth shall not be less than three inches.

Diameter of Bars.—73. The least diameter or thickness of independent reinforcing bars in slabs shall not be less than one-quarter of an inch.

Wires.—74. The wires under tensile stress in connected mesh and similar reinforcement in slabs shall be at least one-tenth of an inch in diameter or thickness.

Space between Bars.—75. There shall be a distance of at least one inch between bars in slabs except at joints and at points where the bars are in direct contact and transverse to one another.

76. All meshed reinforcement shall be of such dimensions as will enable the coarse material in the concrete to pass easily through the meshes of such reinforcement.

* NOTE.—Five inches is the least thickness for fire-resisting floors in the principal Acts.

77. The maximum distance between bars or strands of the tensile reinforcement in slabs shall not be greater than 12 in., and not more than twice the effective depth of the slab.

Distributing Bars.—78. In cases where independent reinforcing bars are provided in one direction only, distributing bars shall be provided on the top of the lower tensile bars at right angles thereto. Such distributing bars shall not be further apart than 18 in., and shall have an aggregate cross sectional area of at least 0.08 per cent. of the effective cross sectional area of the slab; or the diameter of each of the distributing bars shall be at least 1/16th of the effective depth of the slab, and the pitch of the distributing bars shall be not greater than four times the effective depth of the slab.

Wiring.—79. Wiring used in slabs for the purpose of holding bars in position shall not be regarded as reinforcement.

RESISTANCE MOMENTS.

Resistance Moments.—80. The resistance moment of reinforced concrete construction under transverse loads shall be determined by formulas based on the following assumptions:—

- (a) All tensile stresses shall be taken by the steel.
- (b) The strain in any layer or fibre is directly proportional to the distance of that layer or fibre from the neutral axis.
- (c) The tensile elastic modulus of steel shall be assumed to be equal to the compressive elastic modulus of that material. (See regulation 53.)
- (d) The elastic moduli of the concrete remain constant within the limits of the working stress.
- (e) The stress-strain graph is a straight line.
- (f) The anchorage, together with the grip between the concrete and steel, is sufficient to make the two materials act together.

Notation for beams and slabs.

Notation—81.

A = area of tensile reinforcement, in sq. ins.

a = arm of the resistance moment, in inches.

B = bending moment of the external loads and forces.

b = breadth of rectangular beam in inches, or breadth of the flange of a tee beam in inches, or breadth of slab in inches.

c = permissible compressive working stress, at the extreme edge of the concrete in compression, in pounds per sq. in.

d_s = total depth of slab in inches.

d = effective depth of the beam or slab in inches, i.e., the distance from the compressed edge of the constructional concrete to the common centre of gravity of the tensile reinforcement.

E_c = elastic modulus of concrete in compression.

E_s = elastic modulus of steel in tension or compression.

l = length of the effective span of a beam or slab.

$m = \frac{E_s}{E_c}$ = modular ratio. (See regulation 54.)

n = distance of the neutral axis from the compressed edge of the constructional concrete of the beam or slab, in inches.

$n_1 = \frac{n}{d}$ = neutral axis ratio, $\therefore n_1 d = n$

p = percentage of tensile reinforcement = 100r.

Q = qualifier in the equation $R = Qbd^2$.

R = resistance moment generally.

R_c = resistance moment of the internal stresses in the beam or slab in terms of the permissible compressive working stress.

R_t = resistance moment of the internal stresses in the beam or slab in terms of the permissible tensile working stress.

r = ratio of A to bd , i.e., $r = \frac{A}{bd}$ and $A = rbd$.

$$s_1 = \text{slab depth ratio} = \frac{d_s}{d}$$

t = permissible tensile working stress, in tensile reinforcement, in pounds per sq. in.

t_1 = ratio of the tensile stress in the steel to the compressive stress at the extreme edge of the concrete under flexure $= m \left(\frac{1}{n_1} - 1 \right)$

W = total weight, or working load.

Beams and slabs.

Breadth of Flange.—82. For the purpose of computing the resistance moment of a tee beam, the breadth of the flange shall not be taken at more than—

- (a) One-fourth of the effective span of the tee beam;
- (b) The distance between the centres of the ribs of the tee beams;
- (c) Twelve times the thickness of the slab

whichever is the least.

(Also see regulations 45 and 46 as to combined stresses in beams, slabs, and other members.)

83. In the case of Γ (ell) beams the breadth of the flange shall not be taken at more than four times the thickness of the slab, provided that where the moments due to the eccentricity of the compression are adequately provided for, the width may be increased, but not beyond one-half of that allowed for $\top$ (tee) beams.

Breadth of Rib.—84. The minimum breadth of the rib of a $\top$ (tee) or Γ (ell) beam shall not be less than one-third the depth of the rib below the slab. (Also see regulation 66.)

85. Rectangular beams shall be secured against buckling whenever the length of the beam exceeds 20 times the least width thereof, or whenever the length of the beam exceeds 30 times the least width measured from outside to outside of the outermost longitudinal reinforcement.

Slab Reinforcement.—86. When a part of a slab is taken as forming part of a tee beam, the reinforcement in the slab transverse to the beam must cross the full breadth of the portion of the slab forming the flange of the tee beam.

Neutral Axis Within the Slab.—87. In the case of slabs, rectangular beams and tee beams reinforced in tension only when the neutral axis is within the slab—i.e., tee beams in which r is less than

$$\frac{s_1^2}{2m(1-s_1)}$$

- (a) The position of the neutral axis shall be obtained from the equation—

$$n_1 = \sqrt{(m^2 r^2 + 2mr)} - mr, \text{ or}$$

$$n_1 = mr \left[\sqrt{\left(1 + \frac{2}{mr}\right)} - 1 \right], \text{ or}$$

$$n = \left[\sqrt{(m^2 r^2 + 2mr)} - mr \right] d$$

- (b) The mean compressive stress in the concrete shall be taken at $\frac{c}{2}$

- (c) The arm of the resistance moment shall be obtained from the equation—

$$a = d - \frac{n}{3}, \text{ or}$$

$$a = d \left(1 - \frac{n_1}{3} \right), \text{ or}$$

$$\text{approximately, for tee beams, } a = d - \frac{d_s}{3}$$

- (d) The tensile resistance moment at every cross section shall be at least equal to the bending moment at that section and shall be obtained from the equation—

$$R_t = tA \left(d - \frac{n}{3} \right), \text{ or}$$

$$R_t = tAd \left(1 - \frac{n_i}{3}\right), \text{ or}$$

$$R_t = trbd^2 \left(1 - \frac{n_i}{3}\right), \text{ or}$$

$$R_t = Qbd^2 \text{ where } Q = tr \left(1 - \frac{n_i}{3}\right)$$

(e) The compressive resistance moment at every cross section shall be at least equal to the bending moment at that section, and shall be obtained from the equation—

$$R_c = \frac{c}{2}bn \left(d - \frac{n}{3}\right), \text{ or}$$

$$R_c = \frac{cbd^2}{2}n_i \left(1 - \frac{n_i}{3}\right), \text{ or}$$

$$R_c = Qbd^2 \text{ where } Q = \frac{c}{2}n_i \left(1 - \frac{n_i}{3}\right)$$

Neutral Axis Intersecting the Rib.—88. In the case of tee beams, reinforced in tension only when the neutral axis intersects the rib—i.e., tee beams in which r is greater than

$$\frac{s_i^2}{2m(1-s_i)}$$

(a) The position of the neutral axis shall be obtained from the equation—

$$n_i = \frac{s_i^2 + 2mr}{2(s_i + mr)}$$

(b) The mean compressive stress in the concrete shall not be taken at more than

$$c \left(1 - \frac{s_i}{2n_i}\right), \text{ or}$$

$$\frac{cmr(2-s_i)}{s_i^2 + 2mr}$$

(c) The arm of the resistance moment = a where

$$a = d \left[1 - \frac{s_i}{3} \left(\frac{3n_i - 2s_i}{2n_i - s_i} \right) \right], \text{ or}$$

$$a = d \left[\frac{s_i^3 + 4mrs_i^2 - 12mrs_i + 12mr}{6mr(2-s_i)} \right], \text{ or}$$

$$\text{approximately } a = d \cdot \frac{d_s}{2}$$

(d) The tensile resistance moment at every cross section shall be at least equal to the bending moment at that section, and shall be obtained from the equation—

$$R_t = tAa, \text{ or}$$

$$R_t = tbd^2 \left[\frac{s_i^3 + 4mrs_i^2 - 12mrs_i + 12mr}{6m(2-s_i)} \right], \text{ or}$$

$$R_t = Qbd^2 \text{ where}$$

$$Q = t \left[\frac{s_i^3 + 4mrs_i^2 - 12mrs_i + 12mr}{6m(2-s_i)} \right]$$

(e) The compressive resistance moment at every cross section shall be at least equal to the bending moment at that section, and shall be obtained from the equation—

$$R_c = c \left(1 - \frac{s_i}{2n_i} \right) b d_s a, \text{ or}$$

$$R_c = cbd_s d \left[\frac{s_i^3 + 4mrs_i^2 - 12mrs_i + 12mr}{6(s_i^2 + 2mr)} \right], \text{ or}$$

$$R_c = Qbd^2 \text{ where}$$

$$Q = cs_i \left[\frac{s_i^3 + 4mrs_i^2 - 12mrs_i + 12mr}{6(s_i^2 + 2mr)} \right]$$

Compressive Reinforcement.—89. In sections reinforced in compression due allowance shall be made for the depth of the compressive reinforcement from the compressed edge of the beam.

90.—The equations in regulations 87 and 88 may be used in conjunction with regulation 61 when the compressive reinforcement is located at or near the centroid of compression in the concrete.

Beams supporting other Beams.—91. In the case of a beam supported at its end by a transverse beam—

(a) Longitudinal bars of the supported beam shall be continued across to the further side of the supporting beam, and the ends of the bars shall be hooked.

(b) The whole shear shall be provided for by the tensile resistance of the shear or web reinforcement acting in conjunction with the compressive stresses in the web.

(c) Provision shall be made to resist any torsion in the supporting beam, and to resist any negative bending moments on the supported beam, but the positive bending moments on the supported beam shall be calculated on the assumption of ends being freely supported.

92. At the ends of beams supporting transverse beams the bars shall be so arranged as to give the same margin of safety as required by regulation 91.

Notes on Drawings or Diagrams.—93. Notes shall be added to the drawings or diagrams of all beams and pillars showing the loads which have been provided for.

MEMORANDUM.

American Society of Testing Materials.—The annual meeting of this Society took place recently, when Prof. Mansfield Merriman was elected President. A number of questions came up for consideration, and reports were presented by the different Research Committees on such questions as rail shrinkages, paint tests, tests on building brick, etc.

Regarding cement and concrete, the Committee's activities were restricted as regards concrete to consideration of the constituent materials of concrete rather than to the finished concrete.

Concrete Aggregates.

The Committee C-9 on Standard Tests for Concrete and Concrete Aggregates (Sanford E. Thompson, chairman) has outlined a most elaborate series of tests which is now under way, the tests themselves for the most part being made in various commercial and college laboratories. Work is being carried on in co-operation with the Committee on Specifications and Methods of Tests for Concrete Materials, of the American Concrete Institute.

Sub-Committees have been appointed, each to direct the investigations into one particular branch of the general subject. There is no better method of conveying the extent of the programme than by abstracting the work of each Sub-Committee as given in the Main Committee report :—

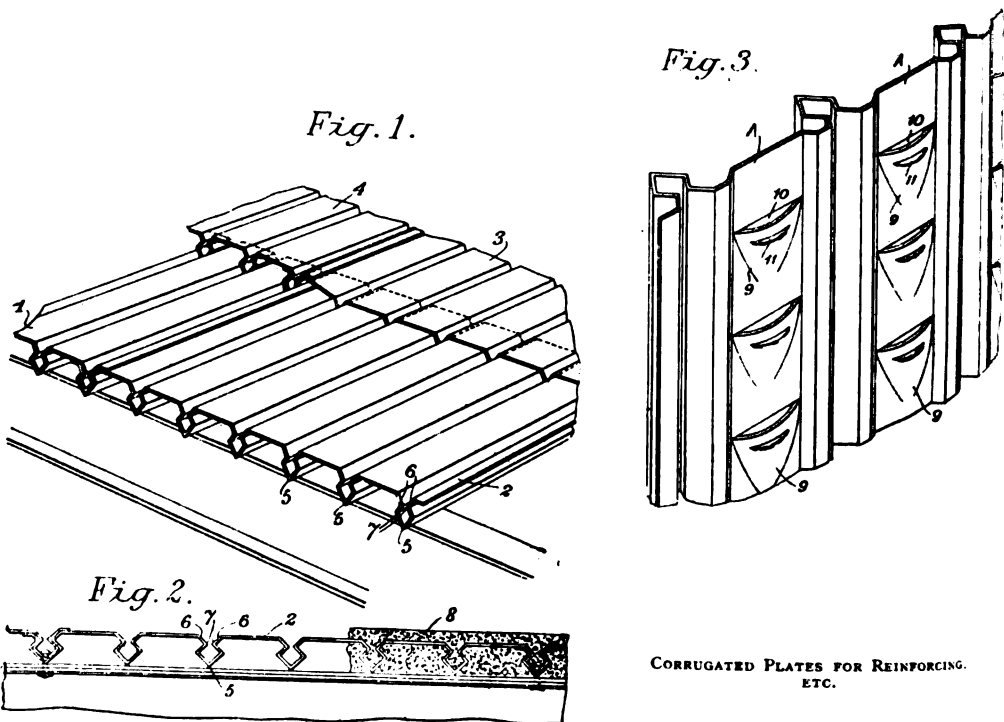
Laboratory Tests for Concrete and Laws of Mechanical Mixtures.—This Sub-Committee has outlined certain tests on concrete and mortars with the view of establishing the best practice in making and testing concrete specimens in the laboratory and to determine the effect of variations in the gradation of sizes of the aggregates on the strength and properties of the concrete. These tests cover a wide range and necessitate the making and testing of a large number of specimens. It is felt that only by making such tests exhaustive can information be obtained which will permit the drawing of definite conclusions. It is hoped that these tests will furnish information upon which can be based definite recommendations as to the practices which should be followed in testing concrete in the laboratory; and it is also hoped that definite conclusions can be reached as to the effects of certain variations in the gradation of the sizes of the aggregate materials. The following series of tests have been outlined as a part of the necessary investigations :

1. Comparison of metal and wood forms for standard laboratory tests of concrete.
2. Variations in strength and properties of concrete throughout the batch.
3. Effect of different methods of tamping and compacting.
4. Effect of condition of storage of specimens for standard compression test of concrete.
5. Method of capping test specimens preparatory to testing.
6. Effect of gradation of sand.
7. Effect of varying ratio of fine to coarse aggregates.
8. Effect of varying sizes of fine to coarse aggregates.
9. Strength of mortar as affected by the form of the test specimens.

RECENT BRITISH PATENTS RELATING TO CONCRETE.

We propose to present at intervals particulars of British Patents issued in connection with concrete and reinforced concrete. The last article appeared in our issue of July, 1915.—ED.

Corrugated Plates for Reinforcing, etc.—No. 4,811/14. Henry Luis Lewen, 220, West 42nd Street, New York, U.S.A. Accepted February 24/15.—According to this invention a plate for use in concrete and plaster work is provided with corrugations of a wedge shape, and a constriction so that the projecting corrugations on the one side of a plate can be forced into corresponding channels on another similar plate, and thus serve to lock the two plates together. The corrugations also enable plates to be locked together longitudinally as well as transversely, and to assist in holding such plaster or concrete, the plates may be provided with punched out portions forming pockets, which also act to stiffen the plates. For forming the corrugations in the plates a special arrangement of dies is described.



The plates (1, 2, 3, 4) are corrugated in such a manner that a wedge-shaped point (5) is formed at the crest of each corrugation, a wide introductory portion (6) and a constricted portion (7) also being formed. In the preferred construction, such as illustrated, the sides of each corrugation first converge from the body of the plate, giving the wide introductory portion (6), then diverge and then converge to the point (5) as clearly shown, forming the constriction (7), the various bends being sharp angles. The plates may be readily interlocked transversely one upon the other, the wedge points (5) of the corrugations on the back of one plate, such as plate 3, readily entering the corrugations or channels on the face of plate 2 through the wide introductory portions, the plate 3 being forced firmly home against the plate 2, so that the wedge corrugations pass through the constricted portions (7) and are securely locked. As shown, plates

may also be locked together longitudinally, the plate 1 being thus locked to plate 2 and the plate 4 being locked on plates 1 and 3. The portions of the corrugations which first converge from the body of the plate forming the wide introductory portions act as guides for leading in the wedge-shaped points when interlocking. The constricted portions (7) spring apart to allow the wedge points of a plate to be forced through and then spring back to their original position, gripping the wedge-shaped corrugation thus inserted. The constricted portions enable plaster or concrete material to be held on the surface of the plate as the material fills the channels completely and the portions (7) serve to retain it on the surface of the plate, as shown in Fig. 2.

To assist further in retaining plaster or similar material on the plates, punched-out pockets, such as 9 (Fig. 3), may be formed in the plate. These pockets are formed in such a manner that the plates are not weakened in any way, the pocket being formed by making a slit (10) and then pressing the metal away from the plate to form a pocket (9). The pockets thus formed serve to stiffen the plate. In the form shown the wall of the pocket is bent inwards above the slit forming a rib (11), which assists in holding the concrete or similar substance, though, if desired, the wall may be bent to form a rib on the outside of the pocket. The plate is preferably recessed, as at A, to provide an additional stiffened portion in which the pockets are formed.

Piles and Caissons.—No. 17,888/14. *Edward Bignell, Lincoln, Lancaster County, Nebraska, U.S.A. Accepted March 25/15.*—This invention relates to piles or caissons with means for delivering jets of water in advance to form an excavation, into which the structure may sink, and along the sides of the structure to lift the displaced material.

The central longitudinal channel (2) of the pile (1) is formed by moulding the concrete over a form, consisting of middle section (4), of relatively thin sheet-iron, and end sections (5) of metal tubing, the parts overlapping to form a close joint.

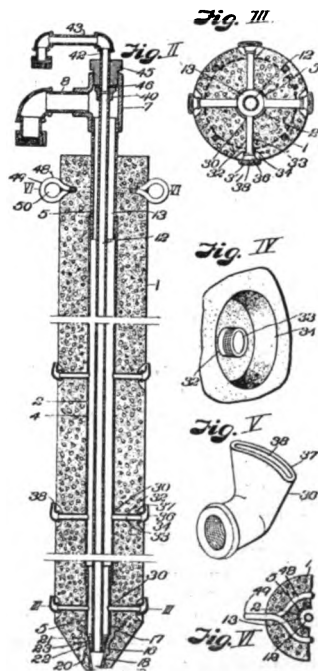
The lower tubing section (5) terminates within the lower end of the pile and is threaded into a head (17) of a nozzle (16) having outstanding ribs (20), and a tapered lower end (18), the end of which terminates flush with the bottom of the pile.

The interior of the nozzle end (18) is tapered, and threaded on to the lower end of the inside pipe (12) is a tapered plug (21), having an anchor groove (22) carrying a packing ring (23), engaging the inner surface of the nozzle to confine water in the space outside the pipe (12) and separate it from water in the pipe (12), so that separate volumes of water under different pressures may be fed to the excavating and lifting nozzles.

The core (4) and tubing (5) are provided with apertures (30) which register with transverse channels (32) in the concrete pile, and fitting within the channels and apertures are tubes (33) which preferably terminate in sockets (34) in the sides of the pile.

The nozzle tips (36), which are threaded on to the outer ends of the tubes (33), have their upwardly directed flared and flattened outer ends (37) lying close to the outer surface of the pile.

On the upper end of the inside pipe (12) is threaded a collar (40), which lies in the fitting (7) above the intake from the supply pipe (8), and threaded into the upper end of the collar (40) is a section of pipe (42) which projects above the fitting (7) to carry the high-pressure supply pipe (43). A gland (45) is slidably mounted on the connecting pipe section (42), and adapted to thread into the upper end of the fitting (7), so that it may seal the fitting and also engage the top of collar (40) to force the inside pipe downwardly in the pile and seat the plug (21); a packing ring (46) is located between the top of the collar (40) and the bottom of the gland (45) to seal the connection at that point.



PILES AND CAISSONS.

Waterproofing Cement.—No. 13,542/14. *Joseph Freeman Goddard, 59, Tytherton Road, Holloway Road, London, N.* Accepted March 11/15.—This invention comprises an improvement upon that of Specification No. 20,782 of 1910, in which a solution of tannin or tannic acid in water is used as a waterproofing agent.

According to the present invention, the tannin or tannic acid is mixed with gypsum or similar substance and added to the clinker before grinding.

The invention is carried out as follows:—

The waterproofing material, tannic acid or tannin, is mixed with gypsum or other suitable form of calcium sulphate or similar material for "slowing" cement in the proportions of 1 part by weight of tannic acid or tannin to 5 parts by weight of gypsum. The materials may be ground either before or after mixing. Sufficient water is then added to produce a paste, and the mixture is allowed to set. It is then ground to a powder, and this powder is used for adding to the cement clinker before the latter is ground.

The amount of the treated gypsum to be added to the clinker will depend on the nature of the latter and on the nature of the cement it is desired to obtain, and particularly on the setting time required, as it is well known that the setting time of Portland cement may be varied between limits by the addition of varying quantities of gypsum. It will generally be found, however, that an addition of treated gypsum corresponding to 0·8 per cent. by weight of the clinker will yield an average slow setting cement. Care should be taken that the temperature of the materials during grinding does not rise above about 300 degs. Fahr., otherwise the tannic acid might decompose.

The following tests are quoted in the Specification:—Discs made up of cement, produced according to this invention, and sand in the proportions of 1 of cement to 3 of sand were prepared, the discs being half an inch thick and about two and a half inches in diameter; similar discs of cement and sand were prepared with cement in all respects similar to the above, with the exception that the added gypsum had not been treated with tannin or other similar material. Both kinds of discs were kept in moist air for 48 hours, then placed in water for three weeks, and afterwards kept in moist air for one week, the temperature of the water and air during the whole of this period of storage being in both cases 15 deg. C. Both discs were then submitted to the action of water under a pressure of 4 lb. per sq. in. Those made of the untreated cement were found to be wet through in 24 hours, whilst in those produced from the treated cement in accordance with the invention the water was found to have penetrated through the upper surface of the discs to a maximum depth of one-twentieth of an inch after one month. Similar, but slightly better results were obtained where the discs were made up with one portion of cement to 2 portions of sand.

Similar results were obtained when discs made of cement manufactured according to this invention were tested for their water resisting powers at a pressure of 52·57 lb. per sq. in.

The following average breaking weights in pounds per sq. in. of net cross section were obtained:—

Time.	Neat Cement.	Neat Cement. (treated).	Cement & Sand.	Cement & Sand (treated).
7 days	629·5	618	339	323
28 days	730	701	374·5	445
90 days	785	804	439·5	645

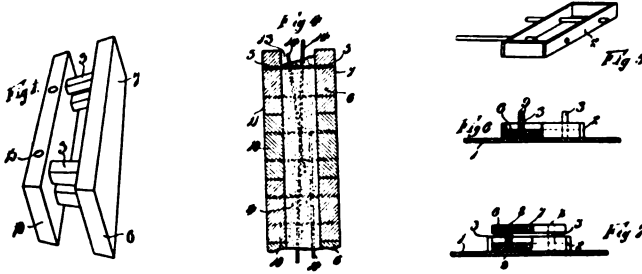
Concrete Walls.—No. 27574/14. *H. M. Archibald, 745, St. Andrew's Avenue, North Vancouver, British Columbia, Canada.* Accepted July 1/15.—This invention comprises improved means for constructing concrete walls formed of concrete blocks of the type in which there is an interior vertical slab (6), Fig. 1, an exterior vertical slab (10), and separately formed horizontal parts (3) connecting the slabs.

In accordance with the invention, one of the slabs is formed first by flowing plastic material into an open frame placed upon a horizontal table, the connecting pieces being placed vertically within the open frame and in the plastic material. When this half-made block is sufficiently set and hard it is removed from the frame and table and then replaced in an inverted position; the other slab is then cast on to it, the other ends of the connecting pieces being embedded in it.

The surface (1) of the table is preferably of plate glass, but slate or enamelled iron

or any other suitable material may be employed, and the surface of the table may be plane or curved. The connecting strips (3) may be bricks or of stone, or may be moulded rods of concrete with or without a metal core, but if it is intended ultimately to fill in the wall solid with concrete, as shown in *Fig. 4*, the connecting pieces should preferably be bare metal (5).

If the connecting pieces (3) are of a light structure, and therefore liable to be displaced by the pouring, it is preferable to pour the plastic material (6) first, and then



CONCRETE WALLS.

push the connecting pieces (3) into the material while it is yet soft. Before pouring the plastic material there may be placed upon the glass table (1) tiles, embossed-paper, glass, or other material, to form the surface of this slab (6).

When this first slab (6) has sufficiently set the frame (2) is removed, and the slab (6), with its embedded connecting pieces, is lifted from the glass (1) and replaced in an inverted position, with the lower ends (9) of the connecting pieces (3) resting upon the glass (1) and the upper ends (8) supporting the completed slab (6), but the half-made block may be inverted before removing it from the frame (2) if desired.

The frame (2) having been replaced on the glass, the material to form the second or outer slab (10) is now poured. As in the other slab (6), the outer surface (11) may be formed of terra cotta, stone, or similar material by placing the latter upon the glass table before pouring. When all the material is sufficiently set and hard the frame (2) is removed.

Into holes (15) in the outer slabs are placed pieces of metal (12) long enough to pass through several slabs at a time; the holes in the space (15) around the metal (12) are filled with Portland cement grouting (16) or other suitable material. The holes (10) are formed by means of removable rods passing through the frame (2), as illustrated in *Fig. 5*.

Reinforced Concrete Strong Rooms.—No. 21352/14. J. Bennett, 16, Deansgate, Manchester. Accepted July 22/15.—In accordance with this invention, strong rooms

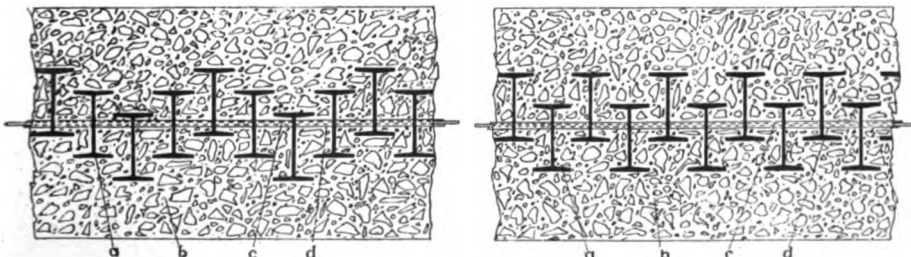


FIG. 1.

FIG. 2.

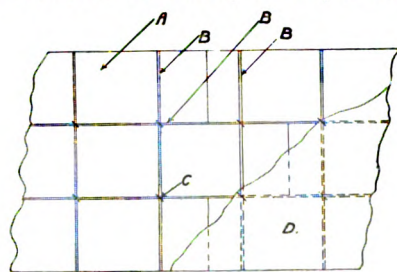
REINFORCED CONCRETE STRONG ROOMS.

and the like are formed of steel joists arranged in zig-zag manner and encased in concrete, the arrangement allowing the joists to be placed close together while enabling the concrete to be tamped round them.

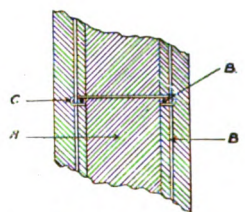
Two arrangements are shown in the figure; tie bolts (*c*) are passed through the webs of all joists, and distance pieces (*d*) used to secure each joist in correct position until the concrete (*b*) has been applied.

The joists are usually fixed at the same centres as the width of the flange, but this may be varied.

Reinforced Concrete Walls.—No. 13046/14. *R. Dudley, 120, Victoria Street, Westminster, S.W.* Accepted May 27/15.—This invention comprises an improved method of construction of reinforced concrete walls, partitions, etc. A core (*A*) of concrete or other hard incombustible material is first made by means of blocks which are keyed, dovetailed, or otherwise joined together. On the outside of the core so formed are fixed on one or both sides horizontal, vertical, or diagonal metal reinforcing rods (*BB*).



—ELEVATION—
FIGURE I



—ENLARGED CROSS SECTION—
FIGURE III

REINFORCED CONCRETE WALLS.

Stirrups (*C*) of wire to meet shear stresses hold the reinforcing rods firmly in their proper positions. The external surfaces of the block wall and reinforcing rods are then covered by hand or mechanical means with one or more coats of a mixture of Portland cement and sand or other gritty material in suitable proportions and to a suitable thickness.

Concrete Buildings.—No. 1865/14. *J. E. Pick, 54, Canterbury Street, Garston, Liverpool.* Accepted July 15/15.—This invention relates to concrete buildings having cavity walls and consists in the construction of collapsible core boxes with an outer case secured to sloped cheeks, and into which tapered cores are placed, the cores being capable of being easily withdrawn to allow the core boxes to collapse. The ends of the walls may be formed with a keypiece, which ensures the interlocking of one section of the wall with the adjoining section when formed.

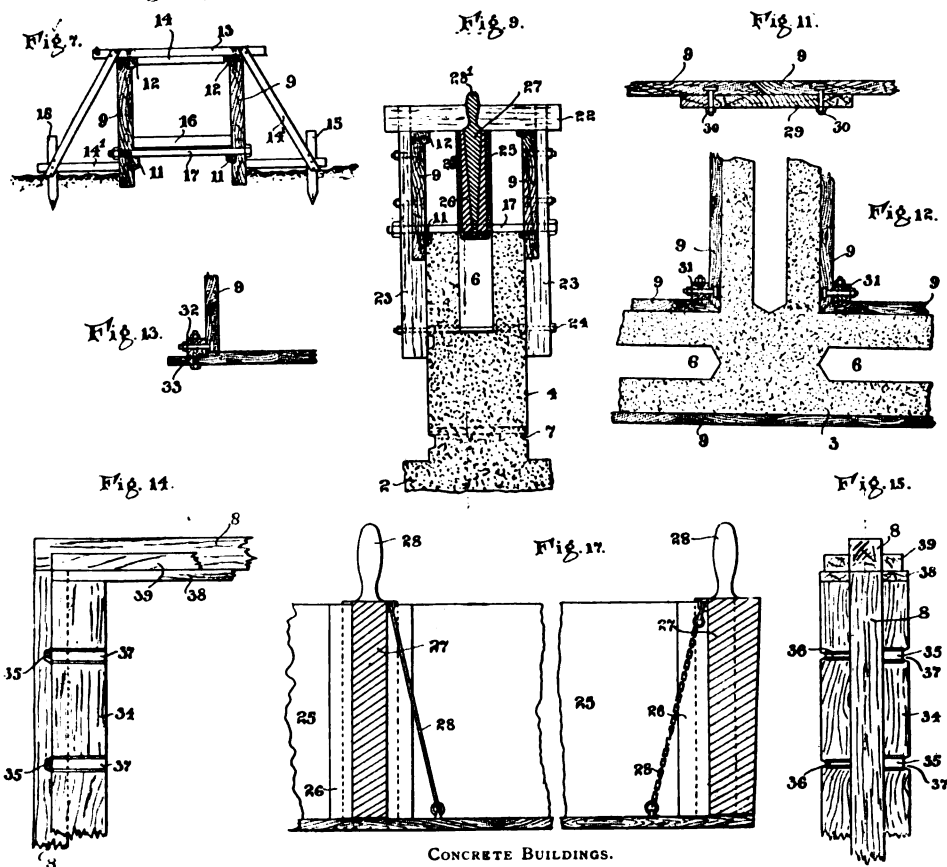
After the foundation of the building has been laid a solid course is erected by means of the sheeting (9), *Fig. 7*, preferably made of timber. The sheeting (9) is provided with the usual fillets (11 and 12), which are adapted to make the grooves and to rest in them when erecting the next higher course. The sheeting (9) is first placed in position upon the foundation and is then levelled and trued and fixed in such position by means of cross-slats (13), which are provided with distance pieces (14) and are secured to the ground by the use of battens (14¹) connected to pegs (15), which are securely driven into the ground. Distance pieces (16) serve to maintain the required space near the bottom of the sheeting, whilst bolts and nuts (17) are passed through the sheeting to prevent it from falling apart.

In the construction of the hollow courses the sheeting (9) is supported in the grooves (7), *Fig. 9*, of the course last finished. Cross-bars (22) are arranged at intervals at the top and are connected to stays (23) passing down the sides of the sheeting and of the lower course, where they are fixed by bolts (24) which are passed through the holes left by the bolts (17) during the erection of the preceding course. The cavity (6) is formed by collapsible core boxes, which are composed of an outer case (25) secured to sloped cheeks (26), into which tapered cores (27) are forced. Each core has a handle (28¹) by which it may be inserted and withdrawn from the box. When the core box is in position, and because of the wedge-shape of the cores, the parts of the box are free to move slightly towards each other when the cores are lifted, and may be then lifted out in any convenient manner, say by hand, or, as an example, by means of the bottom boards, to which some lifting means, such as wire loops, chains, or links (28), *Fig. 17*, are attached.

The case (25) may be secured to the sloped cheeks (26) in any suitable manner, as by hinges at the lower end, so as to permit the parts of the box to move towards each other when the cores are withdrawn.

In cases where two members of the sheeting have to be joined end to end the sheeting is made in two parts, which are joined by a strap (29), *Fig. 11*. Such joint is preferably in the centre, and the two parts of the sheeting can be used as if it were only one. When removing the sheeting it is only necessary to unscrew the nuts (30) and to remove the straps (29); the sheeting can then be easily taken away.

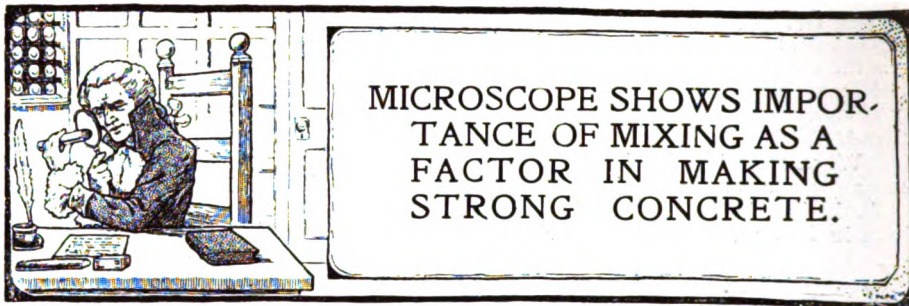
At the junctions of the outside wall and the partition the sheeting may be joined as shown in *Fig. 12*, in which the ends of the sheeting (9) are bevelled and are secured



CONCRETE BUILDINGS.

together by strips (31). In a similar manner the joints at the extreme corners of the building are made by a strip (32) fitted into the angle between the projecting end (33), *Fig. 13*, of one member of the sheeting (9) and the end of the adjoining member.

In addition to the horizontal sheeting (9) vertical sheeting will be required at the door and window openings. The uprights (34), *Figs. 14, 15*, have a longitudinal groove for the jambs of the door or window frames (8). They are provided with fillets (35, 36) to form grooves in the end of the wall at the door or window, and they also have grooves (37) in their sides which enable the sheeting (9) to be erected without the vertical sheeting interfering in any way. The two uprights which are used for each window or door are joined at the top by boards (38, 39), and the top transom of the frame (8) is fixed at both ends to the jambs of the frame, the top transom being left in position after the door or window frame is completed whilst the sheeting is removed.



Size of Aggregate and Quantity of Water Used Also Influence Strength of Mixture.

By NATHAN C. JOHNSON,
Consulting Engineer, New York City.

Fifth article of series.—In reprinting these articles we would state that the opinions expressed are those of the author, and the Journal as such does not necessarily associate itself with the conclusions arrived at. The articles are reprinted by the courtesy of the "Engineering Record," U.S.A., and the illustrations have been placed at our disposal by the author.—ED.

IN Article 3 the proportioning and grading of aggregates was shown to have a pronounced effect on the quality of concrete. Sections taken through specimens moulded from mixes of varying proportions showed varying stone content and arrangement, with consequent greater or less thickness of cement layer. The necessity of proportioning cement according to the burden imposed by each particular aggregate was emphasised by reference to these test results. However, all difficulties with concrete cannot be blamed upon improper proportioning of aggregate. There are other factors equally vital which receive as little, or even less attention; and these are to be the subjects of the present paper.

STONE AND CONCRETE COMPARED.

It has been previously stated that a product having a density equal to that of natural stone is an ideal concrete. It has been shown also that the high density of natural stone is due to close compacting of the components, with the cementing substance between them present only in the very thinnest layers. On the other hand, it has been shown that the component particles, large and small, of our inferior artificial concretes are widely separated; and that the layers of cementing substance—usually the weakest element of the combination—are correspondingly thick, with a proportionate percentage increase of air and water voids in the mass.

In order to emphasise this difference a microphotograph of a natural concrete (Medina sandstone) and another of an artificial concrete, the latter made under U.S. Government supervision, with graded aggregate, and presumably with the best materials and by the most approved methods, are placed side by side in Figs. 22 (a) and (b). Both photographs show equal magnification—200 diameters. No effort was made to exaggerate differences by choosing a poor section of surface for the concrete or a good one for the stone. Rather, if any partiality was shown, it was in favour of the concrete.

CEMENT COMPONENT.

With this comparison in mind consider for a moment the qualities of the cement component of any concrete. Of all the ingredients cement is the most expensive and the most subject to chemical attack. Further, in most concretes, it is inherently the

weakest portion, and the paste of cement and water is *the only part in which injurious air can be entrained*. Obviously, the less there is of it, beyond a certain necessary minimum, the stronger and the cheaper will be the concrete.

Neglecting other factors for the present and considering only the strength of the cement matrix, the reduction of the thickness of cement layer by closer compacting of aggregates has the same effect as shortening a column of given section, *i.e.*, it will stand a greater load per unit area. This effect is well known; and a curve for sandstone, but characteristic of other materials as well, giving the relation between load sustained and the ratio of height to sectional area, is shown in *Fig. 23*, which is taken from Upton's "Structure and Properties of the More Common Materials of Construction." As this ratio of height to area approaches zero the strength approaches infinity, so that the thinner the layer the greater its supporting value, and the thicker the layer, up to a certain point, the lower its supporting value.

This truth has an interesting and easily demonstrable application in cement testing. Varying thickness of cement layer is probably the main reason that standard

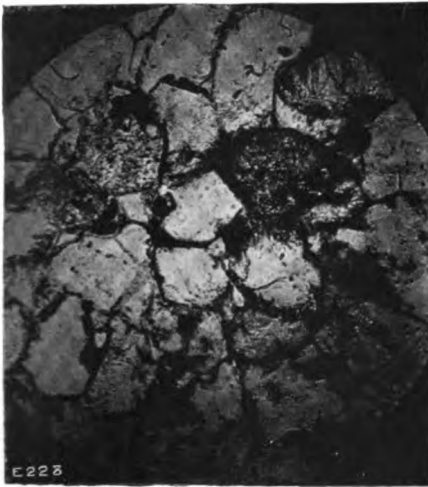


Fig. 22a. Medina Sandstone.

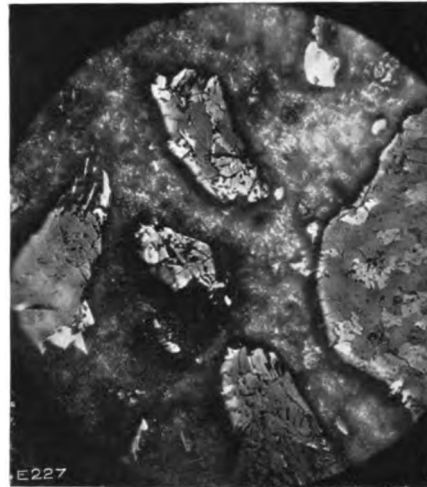


Fig. 22b. Concrete from Hawaiian Pier.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

tests of the same cement and the same aggregate made by different operatives are usually so discordant as to be practically useless as bases of comparison. Different men possessing different degrees of muscular strength produce different thicknesses of the cement layer between the sand grains, or, if the expression be preferred, produce closer compacting of the sand in the briquettes, with the result that a certain allowance has always to be made for the "personal equation." This also relates back to the superior strength shown by the Fuller's curve concrete in Article 3, in which increased percentage of graded stone particles necessitated thinner layers of cement matrix. But assuming proper grading and proportioning of materials, there is yet much to be accomplished in producing good concrete.

Theoretically, at least, if not actually, the dry ingredients—sand, stone, and cement—are capable of forming a dense mass by the finer particles filling progressively the spaces between the larger. But the ultimate positions of even the most perfectly graded sand grains and stone particles in a concrete are dependent upon the manner of its mixing and placing. In just what manner these affect the final result is, so

far, a matter of conjecture only. To discover the effects, the purely physical actions involved in wetting a dry substance with water will be investigated.

SHAPE OF WATER PARTICLE.

The physical characteristics of a piece of stone, or of a grain of sand, or of a

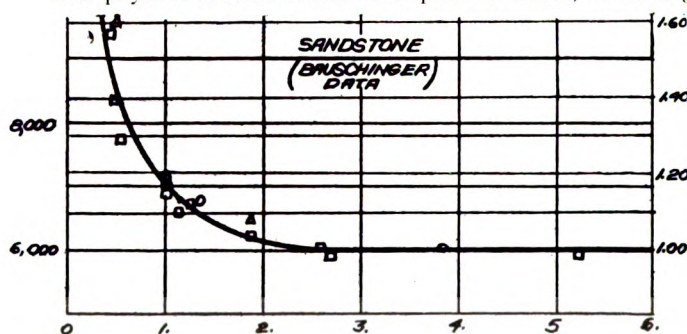


Fig. 23. Relation of Compressive Strength to Thinness of Section.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

particle of cement, are quite generally known within reasonable limits, but the form of a free particle of water may be less familiar. The spray from an atomiser, projected vertically and falling on a microscope slide, is shown in Fig. 24, magnified about 15 diameters. It is evident that each water particle, large or small, is a sphere, or that each at least tends to assume the spherical form. Further, the surfaces of each spherical, or spheroidal, particle must resist gravity to maintain that form, as the centre is obviously higher than any of the other portions. To judge from the tucked-in look of the "edges," where the particle meets the glass slide, there might almost be a tough skin on each particle holding the interior from flowing away. Certainly there is pressure against the bounding surface from within, so that there must be a corresponding pressure inward from without to maintain equilibrium. So much is self-evident.

Curiosity as to the effect of such water particles mixed with sand particles prompts further investigation. Some dry sand (Standard Ottawa sand) of known size put on a microscope slide and tapped to secure close packing, shows as in (a) Fig. 25. Using the atomiser as before, but catching the drops on the slide covered with sand, shows no effect at first, but when the water drops are in sufficient number to unite they do it with such a rush that the sand grains, weighty as they are, are pushed and pulled much closer than it was possible to place them when dry. Further, they are held tightly in place by that stressed "skin" of the big water drop inside of which they lie, as in (b) Fig. 25, so that it is difficult to move them. If, however, the miniature rain from the atomiser is continued, there comes a time when the centre of the water sphere has risen to such a height that its weight is greater than the resistant pressure; and when this occurs the drop disperses and the sand grains rush apart, singly or in groups, with as much intensity as they previously came together, so that no pushing or pulling can arrange them as they were a moment before, as the water itself now insists on occupying space between them. This is shown in (c) Fig. 25.

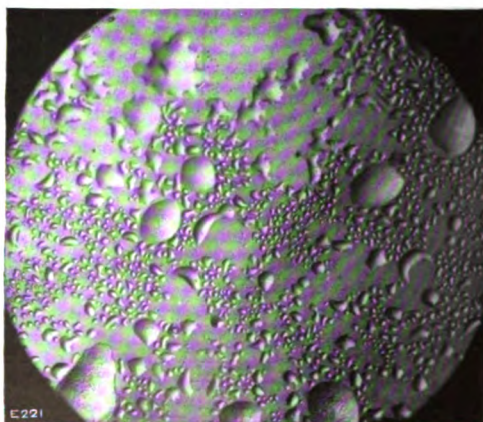
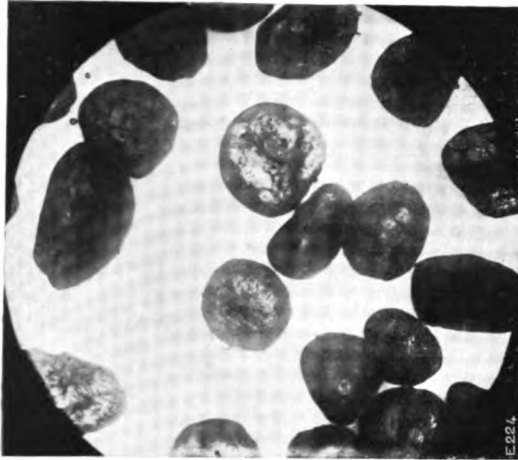
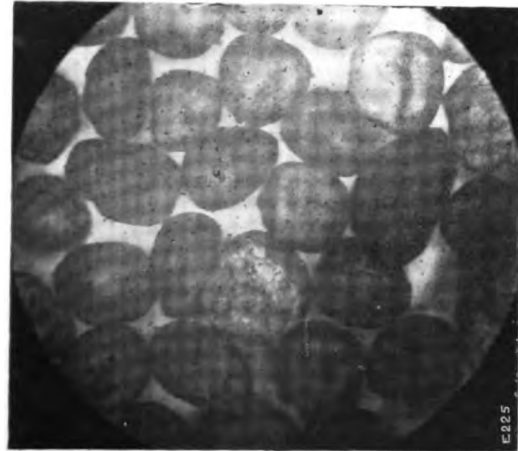


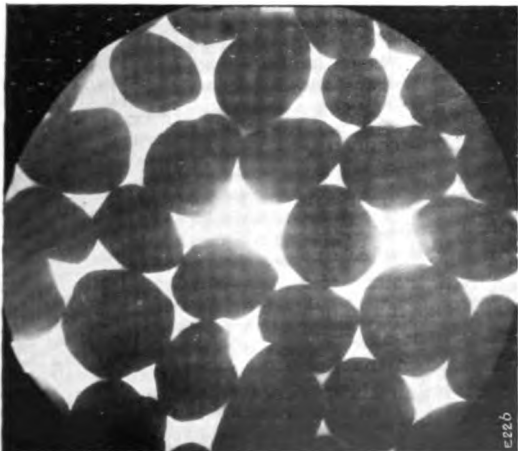
Fig. 24. Water Sprayed by Atomiser: 15 diameters.
THE MICROSCOPE IN THE STUDY OF CONCRETE.



(c) Sand and excess water.



(b) Sand and little water.



(a) Ottawa Sand, dry.

FIG. 25. Microscope shows Effect of Small and Large Additions of Water to Dry Sand; 15 diameters.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

SURFACE TENSION.

This tendency of water to assume the spherical form is universal (as witness rain drops); the interaction of forces that produce this effect is known as "surface tension." Further, as its name implies, there is an actual *surface tension*, or "skin effect," exhibited by the water, and in mixing concrete, whether by hand or by machine, in batches large or batches small, it is constantly in action. This is one of the main causes for the separation of aggregate, the cause of which was sought. If taken advantage of the surface tension effect may be of great use in concrete mixing, but if disregarded it proves detrimental to an equal, if not to a greater, degree.

In the foregoing demonstration Standard Ottawa sand, although rather coarse, was chosen because it is graded to known size. If the effects thus exhibited on standard sand were taken as a basis of comparison, it is obvious that particles of lesser size, such as fine sand grains, would be more easily moved, and that particles of greater size, such as large sand grains, or stones, would be less easily moved by any given force.

VOLUME OF SAND WITH DIFFERENT PERCENTAGES OF WATER.

This has an everyday application closely related to concrete mixing, in the compacting of sand of different grades. The relation between volumes and percentages of water for sands of varying

degrees of fineness, as determined by experiment, is shown in Fig. 26, taken from the report of the Chief of Engineers, U.S.A., 1895, page 2935. It is hardly necessary to point out that the compacting is most marked with the finest materials, or that both the fineness of the material and its moisture content bear directly on the proportioning of concrete. Obviously, the more compact a material the greater the quantity

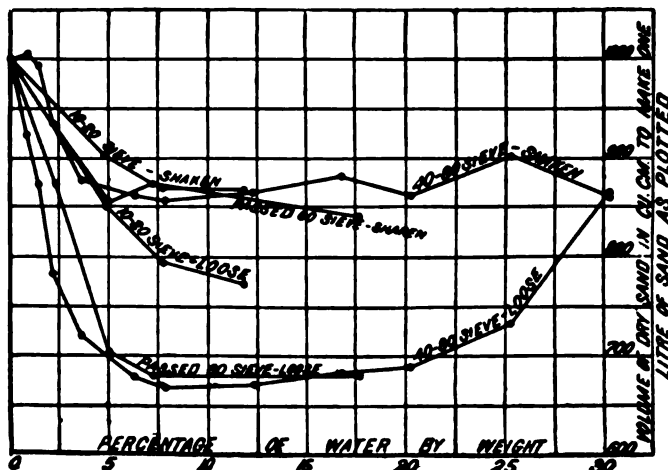


FIG. 26. Relation of Volume of Sand to Water Content.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

that will be contained in a given measure, such as a box or a wheelbarrow. Further, a moment's thought brings the conviction that this surface-tension effect, being most marked with fine materials, must bear directly on the grouping of cement particles which has been an observed characteristic of the concretes thus far studied. It must also largely determine the internal structure of

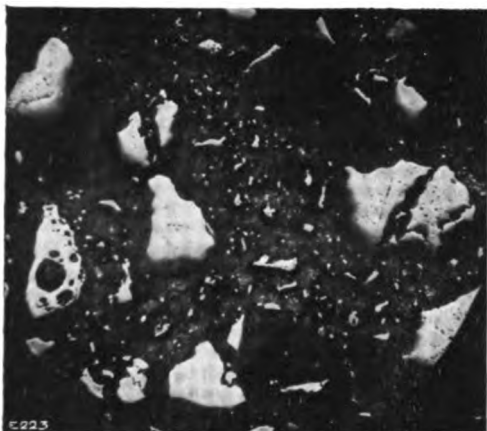
a concrete made with fine sand; and, corollary to this, the amount of water used in mixing must have a very pronounced effect on the quality of the concrete. These are new complications not usually considered, but they may prove worth investigation.

Consider, first, the use in a concrete of a very fine sand or a non-adhesive silt. The material might reasonably be expected to be of great value in producing a dense concrete, both by filling voids and by stopping minute pores. It is, however, frequently the case that fine materials, such as these *remain wholly untouched by cement*. The initial addition of water brings the fine particles closely together in the manner already demonstrated, and either subsequent additions do not reach these groups in sufficient quantity to exert a dispersive effect or else *the mixing process is cut short*, so that mechanical dispersion is not effected by the grinding action of the coarser materials in the mixer. The result is that these groups, often constituting a very considerable portion of the mass, are held in place only by the confining action of adjacent portions, which, in turn, may themselves be either wholly or partially composed of uncemented materials. In such cases the fine particles have doubly failed of their purpose, for not only are they not distributed as void and pore fillers throughout the mass, but they remain wholly without contact with cement, so that they constitute a rotten spot in the concrete.

This condition is not fanciful. It is frequently met with in field concretes, though it is not often recognised. Fig. 27 (a) shows a microphotograph of such a concrete, taken from a chimney foundation built by a reputable and painstaking firm of engineers. Fig. 27 (b) shows a concrete of similar nature taken from a bridge pier, put in by an equally conscientious contractor. The magnification used in both cases is but 15 diameters, in order to show as much of the aggregate as possible in the limited field of the microscope, but in each it will be seen that the matrix is practically gone from between the larger sand particles; and that the pockets where the finer sand grains lay before they fell out, by reason of being quite uncemented, are distinctly

visible. It may be added that the writer knows of no way of determining the presence or absence of such conditions as are here shown so readily as with the microscope, with the incidental preparation of grinding and polishing the specimen. Other methods of observation obscure rather than reveal defects of this character.

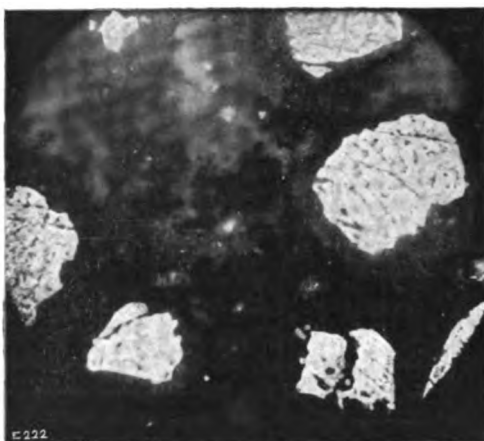
It will probably be said that these are unusually bad cases, especially chosen to emphasise the worst possible conditions, and that proper care would have rendered such results impossible. If such were the case it would be a distinct pleasure to acknowledge it, but it is, unfortunately, a matter of first-hand knowledge, not of conjecture, that similar conditions are only too prevalent, not only in inferior concretes, but in costly and important structures as well.



* Fig. 27a. Concrete from Stack Foundation.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

practice of mixing concrete an insufficient length of time. Unlike the relatively small groups of cement particles, these larger groups of sand grains can be broken up by the mechanical agitation of mixing, provided the process is continued for a suitable length of time. It is often the case that, in the undue haste of supposed efficiency, concrete remains in the mixer drum for only 40 seconds, or perhaps for even less, with sometimes the raw materials pouring in one side before the "mixed" materials have cleared; and because the output looks as much like grey mud as the inspector deems sufficient, it goes into the forms in a final aerating splash that augurs poorly for the completed structure. Eternal vigilance and attention to details are the prices of success in concrete work.

The question of consistency is also a vital one, though, as a general rule, little or no attention is paid to it. A mement's reflection should convince anyone that the amount of water present in a concrete must have a pronounced effect upon its quality. All of the water added must occupy space in the mass unless used in hydration or evaporated or drained off. A certain quantity is required for chemical union with the cement; a certain quantity is necessary to wet the surfaces of the aggregates; and any quantity beyond these two, plus a certain percentage necessary to render the mass sufficiently fluid for convenient handling and proper contact with moulds and reinforcement, is a distinct



* Fig. 27b. Concrete from Bridge Foundation.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

* Magnification, 15 diameters. Black, foggy spaces show where particles untouched by cement lay before falling out.

[illegible]

The importance of proper consistency is quite susceptible of proof. Even if all other factors affecting quality and strength were inoperative the water voids due to excessive wetness are sufficient of themselves to exercise a most pronounced effect. *Every 62½ lb. of water weight in concrete occupies 1 cu. ft. of space; and every cu. ft. of water lost by evaporation or drainage means 1 cu. ft. of voids remaining in the mass.* In Fig. 28 is shown a series of curves plotted from data obtained by the American Concrete Institute from experiments conducted at several universities. The difference in strengths of wet and dry concretes is very striking. (See also H. F. Porter, *Cement Age*, June, 1912; M. O. Withey, "Permeability Tests of Gravel Concrete," *Engineering News*, October 22nd, 1914, p. 811.)

So far as the writer's investigations have gone the effect of surface tension, both in the ways outlined in this article and in others not mentioned, seems to be one of the principal detriments to the production of strong, dense concrete. The effectiveness of the entire mixing process is essentially a function of greater or less differences of surface-tension between the constituent materials, and by proper attention to this factor decidedly superior results have already been produced, with, it is hoped, better ones shortly to follow.

SUMMARY.

Summarising this article it should be noted :

1. That the difference between natural and artificial concretes in thickness of cementing layer, or, if it is preferred, the compacting of aggregates, is extreme.

2. That since the cementitious constituent is the most expensive, the most subject to attack, and (as a paste with water) the only constituent of concrete in which injurious air can be held, the less there is of it, beyond a necessary minimum, the better will be the concrete.

3. That such thickness of an inherently-weak layer is very undesirable in a structural material, as the thinner such a layer can be made, in proportion to the area under stress, the more nearly will it approach in strength the strength of the surfaces by which it is confined, *i.e.*, the strength of the aggregate, and the more homogeneous and the better will be the composite product.

4. That this fact demonstrates the reason for dissimilar results being obtained

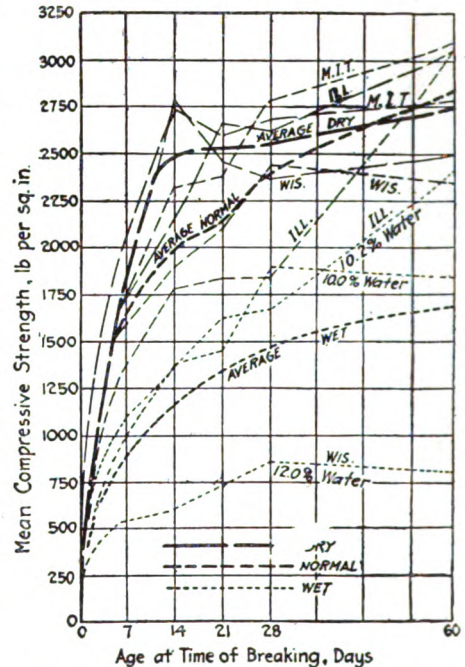


Fig. 28. Time-Strength Curves for Concretes Mixed with Different Percentages of Water.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

in cement work with tests of the same materials by different operatives, *i.e.*, the reason for the "personal equation."

5. That among the causes of this universal wide separation of aggregates in concrete the physical actions involved in mixing a dry substance with water are the chief.

6. That a dry substance, such as sand, when moistened, compacts more closely than when dry, until a certain percentage of water is added, because of "surface tension"; but that when this percentage of water is exceeded the action is reversed and dispersion results.

7. That the finer the material the more pronounced the effect of surface tension; and the coarser the material the less pronounced the effect.

8. That because of this action very fine aggregate lumps together and remains quite untouched by cement, so that weak spots of uncemented materials are frequently found, even in concrete placed in the most important structures.

9. That this grouping can be broken up to a greater or less extent by prolonging the mixing operation, but that where materials are mixed for only a few seconds in order to obtain an abnormal output, inferior, if not positively defective, concrete must be the result.

10. That consistency of mix is a vital matter, though commonly given little attention, as not only is the compacting of aggregate prevented by an improper amount of water and undue thickness of weak material obtained, but further evils are introduced thereby, such as unduly large voids, excessive laitance, shrinkage cracks, segregation of materials of different grades, and the washing away of cement.

(To be concluded.)

MEMORANDA.

Improvements at Blyth Harbour.—Some important extensions are being carried out at Blyth Harbour. The scheme comprises new staiths in the upper harbour which will provide two coaling berths for ships up to 10,000 tons. The staith jetty will be about 1,570 ft. long. Nearly two-thirds of a large tidal basin has been completed. A new turning basin, 9 acres in extent, is being formed almost entirely out of solid rock. A new west pier is also in course of construction. At the termination of the war it is proposed to extend the east pier by the construction of a monolithic concrete pier 1,120 ft. long. This pier will be founded on rock, and its base will consist of concrete blocks, each of about 100 tons weight and built up to a level of 3 ft. to 4 ft. above low water.

Upon the block foundation a monolith of concrete will be raised, as mass work within timber casing, to a height of 7 ft. above high-water level, and it will be surmounted by a decking carried on open framing of reinforced concrete. The extension of the piers is rendered necessary, inasmuch as with only the existing piers it is not practicable to maintain a depth of more than 20 ft. in the entrance channel at Blyth.

The various works enumerated in the foregoing are being carried out to the designs prepared by and under the direction of Messrs. J. Watt Sandeman and Son, of Newcastle-upon-Tyne, the supervising engineers of the Blyth Harbour Commission, and Mr. G. D. McGlashan, M.Inst.C.E., is the resident engineer at Blyth.—*From the "Engineer."*

FIXING TEMPORARY BLINDS TO REINFORCED CONCRETE BEAMS.

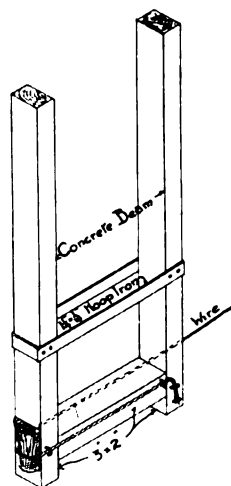
*The following interesting particulars and illustration have been sent us by
Mr. F. Dyer, P.A.S.I., York.—ED.*

In adapting two reinforced concrete warehouses, within a danger zone, for the purpose of temporary barracks, it was necessary to obscure the windows. Two warehouses, ground and first floor, were in use, the walls being formed of sliding doors having a continuous row of glass 38 in. deep along the top of the doors.

The only fixing available was to the main concrete beams, and was accomplished by a clip bracket as shown in the sketch. The legs and strut are of 3 in. by 2 in. redwood, out of ends spared from other work, $1\frac{1}{4}$ in. by $\frac{1}{8}$ in. hoop iron, fixed with clout nails, is put round the legs, the latter put in position, and the strut wedged tightly in and nailed.

The bays are 24 ft. centres, and curtains of dark blue casement cloth 42 in. deep were used, ringed at 10 in. centres on the top edge, and threaded on to iron telegraph wire. Staples to act as eyes and also to fasten the wire were driven into the brackets, and a half-round wedge inserted to take up any slack; probably owing to the wire used there has been no necessity to use the wedges.

There is sufficient friction between the clip and the concrete to carry a considerable load, though it was not possible to make an actual test.



FIXING TEMPORARY BLINDS
TO REINFORCED CONCRETE
BEAMS.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

INSTITUTION OF MUNICIPAL AND COUNTY ENGINEERS. POINTS RESPECTING REINFORCED CONCRETE FOR ROADS, SEWERAGE, etc.

By ARTHUR E. COLLINS, M.Inst.C.E., City Engineer of Norwich.

The following is a short abstract of a Paper recently read at the Annual Meeting of the Institution of Municipal and County Engineers.

Generally.—The ease with which the best shapes for any particular construction can be obtained in reinforced concrete has always made it a fascinating material to engineering and architectural designers. Its durability has been proved in many ways, including the life of reinforced concrete houses built in Newcastle-on-Tyne half a century or more since. When the brick walls of the sea lock at Lowestoft were pulled down some years since, the writer saw hoop-iron reinforcing taken out quite free from rust. It had been in position between high and low water for half a century or more; its condition was due to the protecting influence of mortar. All the reinforcing iron found was in equally good state. That reinforced concrete is reliable is shown by the large number of bridges, reservoirs, piers, wharves, sewers, and buildings of all sorts constructed of it, and giving good service. Where failures have occurred good reasons have been found.

In the early days of reinforced concrete too much reliance was placed on the surface adhesion of concrete to steel, and neither sufficient knowledge nor care was exhibited in properly disposing the reinforcement, nor in making and using the concrete. Accumulating experience has largely remedied this, and to-day good methods are well known and generally applied; but it depends on the responsible engineers of works to ensure that they are applied.

Anyone likely to require to design work in reinforced concrete should study the London County Council Draft of Regulations to be made under the provisions of sec. 23 of the London County Council Act, 1909, with respect to reinforced concrete, dated County Hall, Spring Gardens, April, 1915. It embodies much of the work of the Joint Committee of the Royal Institute of British Architects, Institution of Municipal and County Engineers, District Surveyors' Association, Institute of Builders, War Office, Admiralty, London County Council, and Concrete Institute, and gives, in a much more concise form than the writer is capable of, a very complete set of working rules for reinforced-concrete design and construction, and excepting for suggestions on a few points no further reference will be made to rules.

The writer was much concerned when he drove his first reinforced concrete piles, several years since, by the extensive shattering of their tops, and by the difficulty of making them penetrate the layer of faggots carrying the railway bank he was operating in across a marsh. The piles had been made several months, but had little strength. On considering the matter it was remembered there had been a month of severe frost, followed by cold weather, commencing shortly after the piles were cast, hindering the setting. The period allowed for setting was increased by the duration of frost, but allowance was omitted for the slow thawing during the cold period following the frost.

The remainder of the piles were not driven until thoroughly set, and no further trouble arose.

About fifteen years since the writer constructed reinforced concrete floors in a large building he was then erecting. To obtain lightness he used furnace clinkers largely in his aggregate. His equanimity was much upset by the forward movement of the whole front wall of the building, four storeys high, upon the asphalt dampcourse, which investigation showed to be due to the presence of sulphur in the clinkers, resulting in expansion of cement in the concrete floors. The movement amounted to less than $\frac{1}{2}$ in., and did not continue, nor have the floors given trouble; but furnace clinkers have not been used since.

When constructing reinforced concrete, see to it that all reinforcement is properly placed, supported, and not displaced by the concreting operations. If this be not done, disaster may easily result.

Probably the best material for the aggregate of engineering reinforced concrete is broken Portland stone. This should be used without screening, but care must be taken to remix the various sizes if they sort themselves in transit.

Road Foundations.—The earliest use of reinforced concrete in road foundations known to the writer was by the president when engaged in road improvements in advance of tramway construction for the Middlesex County Council. In one of these roads the writer visited with him the regrading necessitated a considerable depth of clay fill; to avoid the worst consequences of settlements he covered this with a reinforced concrete raft. Upon recent inquiry the writer ascertained this had been successful. The reinforcement did not consist of fencing wire, which is now advertised as sufficient, but of moderately heavy round bars—probably $\frac{3}{4}$ -in. diameter—laid about 1 ft. apart each way, embedded in the concrete.

The writer has had no experience with the use of the meshed wires similar to fencing wire now sold for concrete road-bed reinforcement.

Pile Construction.—Reinforced concrete is particularly adaptable for pile construction, whether they are cast before driving or the mould is prepared in the ground by driving a shell, the reinforcement inserted, and the pile cast *in situ*, withdrawing the shell simultaneously with the casting. Large numbers of piles cast *in situ* are not reinforced.

Manholes, etc.—Reinforced concrete is a most adaptable and suitable material for constructing manholes, underground chambers, etc., in unstable or water-bearing ground. The structures can be mounded in the dry and sunk to the required depths, necessitating the minimum of timbering and pumping.

About twenty years since the writer sunk a large double screening chamber to many feet below water level in very bad, freely permeable ground. The work was quite successful in construction, and has cost nothing to maintain.

Sewer Construction.—The writer has used reinforced concrete largely in sewer construction, and with great economy. The 36-in. pumping rising main at Norwich was constructed in this material some ten years since on the "Bonna" system. It has been entirely successful in all respects. The pipe cost about one-half that of a cast-iron pipe and three-quarters that of a steel pipe. It has cost nothing in maintenance, and appears as good as new.

There are considerable lengths of reinforced concrete gravitating sewage and surface water sewers in Norwich from 42 in. diameter down to 21 in. Some have been laid a quarter of a century. All have been entirely satisfactory; no repairs have been done to any of them, and they are as good as when laid.

Culverts.—A cheap and convenient combination of concrete with steel can be used for the construction of culverts as follows :—

Prepare a suitable concrete bed, finishing to about 1 in. below invert level; on this bone in concrete supports, say 3 in. by 3 in., by about 1 in. high, to the desired gradient and height to support sheet iron or steel slip-jointed pipes; cover the pipes with concrete to the desired thickness. Using suitable concrete, the construction may be buried the day after construction. Eventually the sheet metal will rust away, leaving a concrete interior surface to the culvert. If it be desired to avoid this, insert a spiral of, say, $\frac{3}{4}$ -in. or $\frac{1}{2}$ -in. wire to fit sheet metal pipe, the pitch of spiral being, say, 9 in., place a collapsible mould 2 in. smaller than the pipe concentrically within it, and run the annular space

full of fine concrete. This is, of course, done before the pipes are laid. Allow for the lap of the slip joints when casting the interior concrete.

The writer has used such construction (omitting the concrete lining) successfully and economically in a trench from which the tide could not at all times be excluded.

THE SOUTH AFRICAN SOCIETY OF CIVIL ENGINEERS.

THE PARABOLIC REINFORCED CONCRETE ARCH.

By A. N. HENDERSON, A.M.Inst.C.E., and A. E. SNAPE, M.Sc., A.M.Inst.C.E.

The following is a short abstract of a Paper read before the above Society at their Third Meeting. The abstract is published by the courtesy of the South African Society of Civil Engineers.

THE use of the parabolic form of arch considerably simplifies the calculations, and it is intended in this Paper to consider this form only. At the same time, arches of circular form will approximate in practice closely to the parabolic shape, especially if they are "flat" arches—i.e., with the rise small in comparison with the span, as is generally the case for reinforced concrete arch spans.

In the case of arches with only two hinges, one at each abutment, or of arches with ends fixed in direction at the abutments, it is impossible to determine the stresses by statical methods, and recourse must be had to the elastic theory.

GENERAL THEORY.

According to this method the stresses are found from a consideration of the strains or small movements induced by the loads. The chief strain in arches is one of rotation and is produced by the bending moment. The fundamental equation connecting bending stresses and strains is

$$\frac{M}{EI} \cdot s = A \text{ where}$$

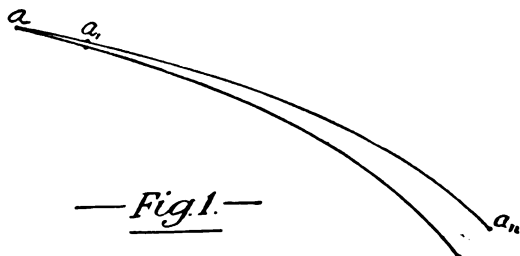
M = uniform bending moment acting on a beam of length s .

E = modulus of elasticity of the material of which the beam is made.

I = moment of inertia of the cross section of the beam about its neutral axis.

A = angle between the tangents at the ends of the beam after bending, the beam being assumed straight before bending.

Consider a portion of an arch aa_1a_n :—



Suppose aa_1 to be very short and of length s_1 and let a bending moment M_1 act on it. If the arch be held fast at a the portion a_1a_n will turn through an angle $A_1 = \frac{M_1}{EI} \cdot s_1$ and neglecting the motion of a_1 itself, which is of the second order of small quantities, the point a_n will move horizontally through a distance $A_1 y_1$, and vertically through a distance $A_1 x_1$, where y_1 and x_1 are the vertical and horizontal projections of the curve a_1a_n ; provided always that the angle A_1 is very small.

* The angle θ in the figures is the same as the angle A in the letterpress.

Consider now the arch ring aa_n :—

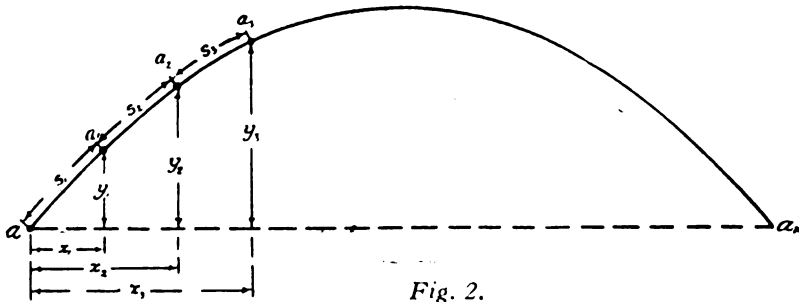


Fig. 2.

Suppose it to be divided up into a number of arcs, aa_1 , a_1a_2 , etc., of lengths, s_1 , s_2 , etc., and acted on by bending moments M_1 , M_2 , etc.

If the arch be fixed at a then :—

$$\text{Total horizontal motion of } a_n = A_1 y_1 + A_2 y_2 + A_3 y_3, \text{ etc.}$$

$$\text{,, vertical ,, } a_n = A_1 x_1 + A_2 x_2 + A_3 x_3, \text{ etc.}$$

$$\text{,, angular ,, } a_n = A_1 + A_2 + A_3, \text{ etc.}$$

But if the arch be also fixed in position and direction at a_n , each of the above quantities is zero. We get then the three equations :—

$$A_1 y_1 + A_2 y_2 + A_3 y_3 + \dots = 0.$$

$$A_1 x_1 + A_2 x_2 + A_3 x_3 + \dots = 0.$$

$$A_1 + A_2 + A_3 + \dots = 0.$$

Substituting the values of A_1 , A_2 , etc., we get :—

$$\frac{M_1 s_1}{EI_1} \cdot y_1 + \frac{M_2 s_2}{EI_2} \cdot y_2 + \dots = 0$$

$$\frac{M_1 s_1}{EI_1} \cdot x_1 + \frac{M_2 s_2}{EI_2} \cdot x_2 + \dots = 0$$

$$\frac{M_1 s_1}{EI_1} + \frac{M_2 s_2}{EI_2} + \dots = 0$$

When s_1 , s_2 , s_3 , etc., are very small these equations become in the notation of the calculus :—

$$\int_0^s \frac{M}{EI} \cdot y \cdot ds = 0$$

$$\int_0^s \frac{M}{EI} \cdot x \cdot ds = 0$$

$$\int_0^s \frac{M}{EI} \cdot ds = 0$$

where S is the total length of the arch aa_n . If the moment of inertia I at any point is inversely proportional to the cosine of the angle which the tangent to the arch ring makes with the horizontal—i.e., to $\frac{dx}{ds}$ we get $\frac{1}{I} = \frac{1}{I_c} \cdot \frac{dx}{ds}$ where I_c is constant and equal to the value of the moment of inertia at the crown. And since E is constant, the equations reduce to :—

$$\int_0^1 My \, dx = 0 \quad \dots \quad \dots \quad \text{Eq. 1}$$

$$\int_0^1 Mx dx = 0 \quad \dots \quad \text{Eq. 2}$$

$$\int_0^1 M dx = 0 \quad \dots \quad \text{Eq. 3}$$

where l is the span of the arch.

These three equations enable us to find the horizontal thrust in the arch and the bending moment at each abutment for any loading, from which the stresses at any point can be computed.

APPLICATION OF THEORY.

Let the arch be parabolic, fixed at the ends, and the moment of inertia such that it is inversely proportional to $\frac{dx}{ds}$ or the arch is "flat" and the moment of inertia is practically constant so that $\frac{ds}{I} = \frac{dx}{I_c}$. Then if any load W act on the arch at any point the reactions at the abutments due to this load will intersect the line of action of the load at a point where a fixed horizontal line, one-fifth of the rise above crown of the arch, intersects this line of action.

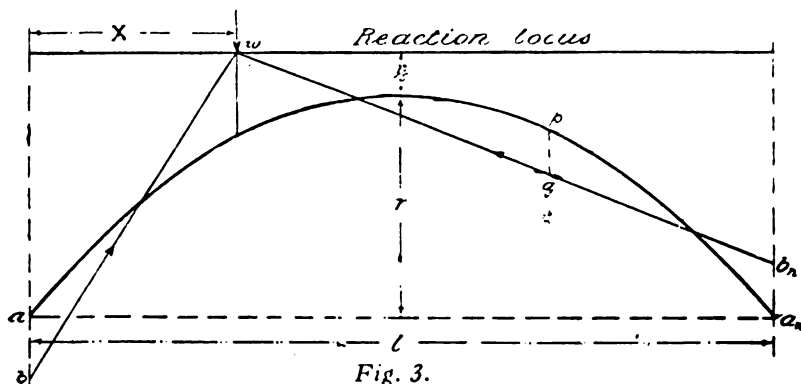


Fig. 3.

In other words, the reaction locus is a straight line, one-fifth of the rise above the crown of the arch.

The directors of the reactions due to the load W are shown in Fig. 3 and are bw and b_nw where

$$ab = \frac{r}{15} \left(10 - 4 \frac{l}{X} \right)$$

$$\text{and } a_nb_n = \frac{r}{15} \left(6 - 10 \frac{X}{l} \right)$$

$$15 \left(1 - \frac{X}{l} \right)$$

The horizontal thrust H is given by

$$H = \frac{15 W.l}{4r} \left(\frac{X}{l} \right)^2 \left(1 - \frac{X}{l} \right)^2$$

and the bending moment at any point $p = H \times pq$ where pq is the vertical distance from b on the central line of the arch to the reaction bw or b_nw .

In all the above equations we have only considered the effect of the bending moment on the arch. The thrust has also some effect as it causes "squeezing" of the central line, but this is small, certainly not more than 5% of the bending moment effect in all ordinary arches, so that it may be neglected without appreciable error.

The author then gave examples to show the application of the foregoing theory to actual design.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

CONCRETE AND ELECTRIC WORK ON NEW YORK STATE BARGE CANAL.

THE accompanying illustrations and drawings show the concrete construction of the New York State Barge Canal, which is largely operated electrically as far as its locks are concerned, and may ultimately be entirely electrically operated from Niagara power and electric energy transmitted along the Canal route from the Long Sault-Hydro Electric Power Development and other power sources located throughout the State.

Concrete Work.—The Barge Canal locks are being built of concrete throughout, both side and cross walls and floor. At a few points, where favourable rock is encountered, the concrete floor has been dispensed with. The side walls are 5, 6, or 7 ft. wide at the top, according to local circumstances, and vary in height and bottom width with the lift of the lock and certain other conditions. In some cases, where one side of a lock is exposed to a river channel, the top width is increased to 12 ft. The lifts range from 6 ft. to $40\frac{1}{2}$ ft. Both the differences of lift and the fluctuations between normal and high navigable stages govern the heights of the side walls, which vary from 28 ft. to 61 ft., with an extreme at one point of the lock at Little Falls of 80 ft.

The bottom widths of these walls, which range between 13 and 34 ft., are determined by the height of the walls, the nature of the foundation and certain incidentals of design at each lock. Unless a rock or hardpan foundation can be obtained, piles are driven under practically all locks. Within each side wall runs a culvert for filling and emptying the lock. The culverts are connected with ports that open into the chamber at the bottom of the walls. These culverts vary in size, the dimensions being 5 by 7 ft. for locks of 12 ft. lift or less, 6 by 8 ft. for lifts between 12 and 23 ft., and 7 by 9 ft. when the lift is 23 ft. or more. Connected with the 5 ft. by 7 ft. culverts are 16 ports, 8 on either side, while the number is increased to 22 with the 6 ft. by 8 ft. culverts and 28 with the 7 ft. by 9 ft. size.

The ports have been made both by imbedding cast-iron pipes in the concrete and by leaving rectangular openings in the walls, the latter being the more recent method. The area of opening in either case is about $7\frac{1}{2}$ sq. ft. each. In some of the locks there is another culvert through one of the side walls—a feature of the hydro-electric development for operating and lighting the locks. Local conditions and the proximity of two or more locks have determined where these power plants shall be placed.

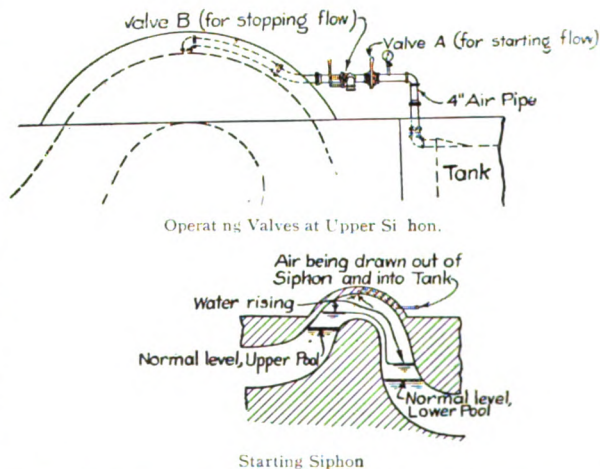


Fig. 1. CONCRETE AND ELECTRIC WORK, NEW YORK STATE BARGE CANAL.

At some points one plant will serve several locks, as at Waterford, where the series of five locks receives power from the plant at Crescent dam. The lock gates are of the mitring, girder type, carrying the principal load as beams. They are built of steel, with single skin-plates, but have white oak quoin and toe posts. The quoin post swings on a cast-steel pivot, set in the concrete, and is held at the top by an adjustable anchorage. The bearing is against cast-iron quoin plates into the side walls. Reinforced concrete power stations, 20 by 30 ft. in plan and about 20 ft. high, are constructed adjacent to the various locks. These stations, with their dark green tile roofs, present a very pleasing appearance.

Electric Work.—There are thirty-three hydro-electric power stations, two gasoline electric power stations, and three sub-stations which supply all the power for operating and lighting the fifty-seven locks of the entire Barge Canal system. Two guard-locks, one on each bank of the Genesee river at Rochester, will be operated by power purchased from an adjacent power company. The hydro-electric power stations, operated by water fed into the canal, are each equipped with two vertical shaft turbines, which in all but a few cases are directly connected to 50-kilowatt vertical-shaft generators, supplying direct current at 250 volts. Where power is furnished to another lock not more than two miles distant, booster sets are installed in the power stations.

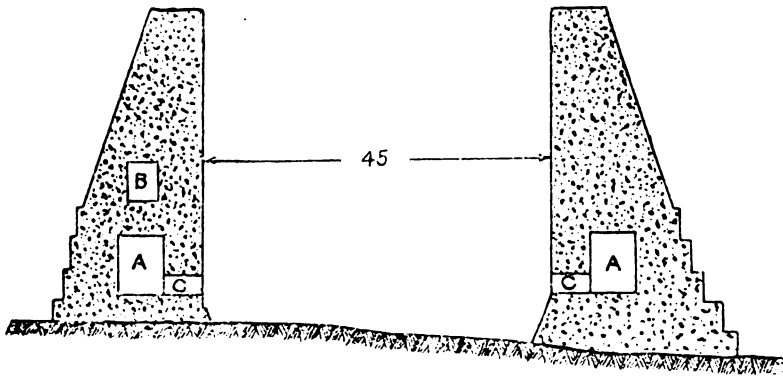


Fig. 2. Cross Section of Lock Chamber—Rock Foundation.

AA—Culverts for filling and emptying lock.
B—Culvert for water-power development.
CC—Port-holes connecting culvert with lock chamber.

CONCRETE AND ELECTRIC WORK, NEW YORK STATE BARGE CANAL.

It is of interest to note that the lock gates are each opened and closed by a steel spar equipped with a heavy coil spring to absorb shock and secured to the gate by a bronze pin. This gate spar is also equipped with a rack actuated by a 7 horse-power motor acting through a train of gears designed to open or close the gates in about one minute. Resistance is supplied in the motor circuit to control the operating speed. A safety coupling, destined to release or break down whenever the load transmitted to the machinery exceeds a prescribed limit, is provided on one of the shafts. Movement of the gates is controlled from operating stands located at each end of each lock on the thrust walls. These operating stands are equipped with master switches of the drum type, by means of which magnet type controllers are set in motion, which automatically regulate the acceleration and speed of the motors.

There are limit switches provided to arrest the motion of the gates at each end of their travel.

There are electric capstans, one at each end of each lock, provided to control the movement of boats along the approach walls and to tow them into and out of the lock chamber. A 20 horse-power motor operates each capstan at a speed of about 60 ft. per minute with a pull of 8,000 lb. The operation of these capstans is controlled by foot switches, placed flush with the top of the lock wall in the capstans. The capstan motors, which are of the mill type and of a specially rugged design, are placed in watertight cast-iron housings, for protection against intermittent wetting surges in the lock or other causes.



FIG. 3. View of some of completed work,
CONCRETE WORK ON THE NEW-YORK STATE BARGE CANAL.

It may be stated that all the motors incorporated in the lock-operating machinery are of the mill type, the armature and field windings of which have been designed to be fire- and water-proof. Telephone systems are installed, connecting the several locks. These systems are so designed and equipped that it will be possible at any future time to extend them, so that a general system along the entire canal will result. Each telephone instrument can be connected to the long-distance system if this is found to be necessary or desirable.

A most interesting feature is the siphon lock. In the city of Oswego there has been constructed a siphon lock—the only lock of this type on the Barge Canal, and the first to be built in this country and the largest employing the siphon principal yet built.

The general design of the culverts is similar to that of a lock of ordinary type, except that at the upper and lower ends the culverts are curved up so as to form necks, which rise a little above the highest water-level and which at the same time are shut off from all communication with the outer air, except through the operating pipes.

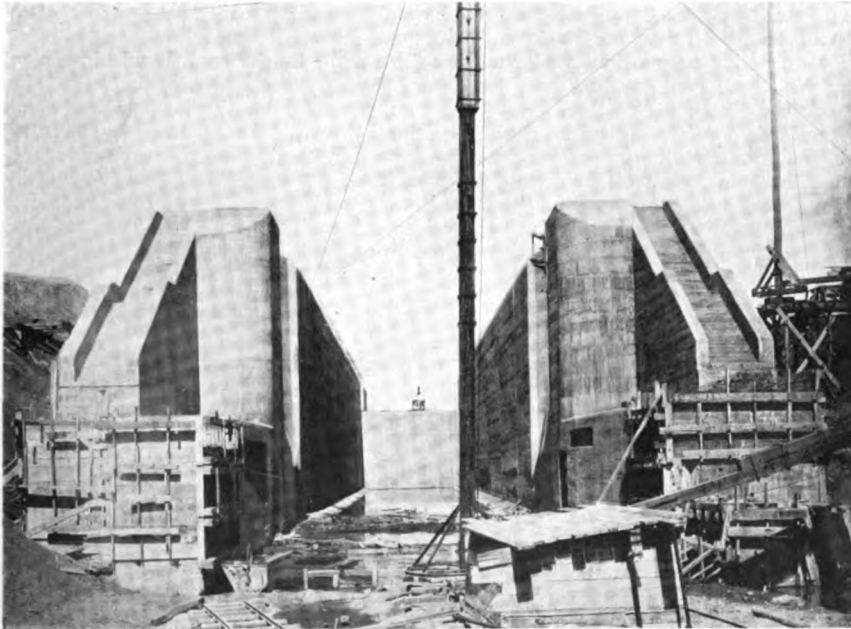


Fig. 4. Lock No. 2.

CONCRETE WORK ON THE NEW YORK STATE BARGE CANAL.

The flow of water is started in the siphons by means of tanks, one being built in each wall near the upper end and communicating through pipes with the upper and lower levels and with both siphons in the same wall and being shut off from all other communication with the outer air. To perform an operation the tank is first filled with water; then the intake valve is closed and the outlet opened. There results a body of water suspended by its weight, but tending to escape into the lower pool, thus producing the necessary vacuum.

On opening the air valve, air from the siphon rushes into the vacuum and water begins flowing over the crest in the neck. Using both siphons the lock chamber can be filled in from $4\frac{1}{2}$ to 5 minutes, while it can be emptied in from $5\frac{1}{2}$ to 6 minutes. It has been found that the draft of the siphon is such that soon after the flow has started the direction of the air is reversed and the vacuum is restored in the tank.

Thus the operating power is self-renewing, and, except for air leakage, lockages can be conducted by merely manipulating the $\frac{1}{4}$ -in. air valves. At Little Falls there has been built a lock notable for its high lift— $40\frac{1}{2}$ ft. The lower gate of this structure is of

the lift type. At two places, Lockport and Seneca Falls, there are combined, or tandem, locks, a flight of two locks at each place. In each case also the normal combined lift is 49 ft.

It may be stated that portion of the old Erie Canal between the Barge Canal crossing near New London and Butternut Creek feeder is to be retained as a navigable feeder, and connection is made with the new channel by a junction lock, 188 ft. long between gates, 45 ft. wide, and with 12 ft. of water. Two other junction locks join the new line to other lateral canals—the Black River Canal at Rome and the Glens Falls feeder at Fort Edward. Also four locks have been built on the Black River Canal where the new Delta reservoir necessitated a relocation of that channel.

In order to maintain navigation on the old canal between Cohoes and Schenectady during Barge Canal construction, a small lock has been constructed at the north end of Vischer's Ferry Dam. This lock will be retained as an auxiliary for the passage of small boats after the completion of the Barge Canal. The Act makes provision for one other lock—where the old Erie Canal enters the Hudson at Albany.

The territory served by canal is very extensive. It appears that 73½ per cent. of the population of the whole State is within two miles of the waterway system and 77 per cent. of the population is within five miles, while 82 per cent. is within ten miles and 87 per cent. within twenty miles. There are thirty-five locks on the Erie branch, eleven on the Champlain, seven on the Oswego, and four on the Cayuga and Seneca. The standard length of lock chamber is 311 ft., and this is the available length for boats. The distances between gates vary with certain conditions, the least being about 338 ft. The locks are 45 ft. wide and have 12 ft. of water over the sills. These locks have lift gates of the same type as the guard gates. The gates are placed 311 ft. apart, and give the same available length as the other locks. They have the usual width and depth of water, 45 and 12 ft. respectively, but no normal lifts.

MEMORANDA.

Dundee, New City Hall.—This new building, which is now in course of erection, is to be built on the most modern principles. Reinforced concrete filled in with stone is to be used for the framework. According to the contract the framework is to be completed by April next. Reinforced concrete piles have been used for the foundations.

Smallpox Hospital, Whitehaven.—The designs selected for this building were those of Mr. H. Irving Graham, of Ellerbanks, Harrington, Cumberland. In the course of a short description of the proposed building the *Building News* says:—The cost of the block is estimated to come to £1,796, without gas and water service, sewage works, road-making, heating system, and fences. The architect says that if reinforced concrete is employed the cost would be reduced by £621, leaving the estimate for the building at £1,175. The laundry block is near the caretaker's lodge. The estimate for this part of the premises is set down at £161, but if reinforced concrete is used a considerable saving would be effected in the cost of the scheme as a whole.

INDUSTRIAL NOTES.

These pages have been reserved for the presentation of articles and notes on proprietary materials or systems of construction put forward by firms interested in their application. With the advent of methods of construction requiring considerable skill in design and supervision, many firms nowadays command the services of specialists whose views merit most careful attention. In these columns such views will often be presented in favour of different specialities. They must be read as ex parte statements: with which this journal is in no way associated, either for or against—but we would commend them to our readers as arguments by parties who are as a rule thoroughly conversant with the particular industry with which they are associated.—ED.

NEW SYSTEM OF INTERLOCKING BLOCKS.

DURING recent years the employment of ready-made concrete blocks has been widely adopted in constructional engineering.

The following are some interesting particulars regarding a new method of interlocking blocks which has been introduced by the firm of Messrs. Peter Lind and Co., Parliament Mansions, Victoria Street, Westminster. The following are the advantages claimed for the system here described:

The blocks are of a special shape and may be used without a structure losing its monolithic properties.

As will be seen from Fig. 1, each block contains a middle portion equal in length to the sum of the two end portions.



— SECTION A-A — — SECTION B-B —

Fig. 1. Elevation and Sections.

The end parts are given the shape of a rail or steel joist, and the middle part takes the reverse shape, the recesses in the former being equal to the projections in the latter. The centre parts thus form fishplates for the joints in the two rows of adjoining blocks.

Further, as the ends of the blocks form hooks (see Fig. 2) the interlocking of the blocks will be so perfect as to transmit both compression, bending and tension circumferentially around the lining just as well as if the structure were monolithic.

A demonstration arch made of these blocks has been erected in a builder's yard at Willesden (Fig. 4).

Each of the segmental blocks weighed 5 cwt. and was handled by four men.

The blocks were placed loosely together without grouting as would otherwise be the case; nevertheless the interlocking device made the construction as strong and rigid as a monolithic structure.

These blocks may be advantageously applied for many kinds of reinforced concrete work.

Tunnel Work.—Fig. 3 shows a section through a tunnel made of these blocks. The lining may form a full circle or have the shape of a horse-shoe, and offers the same strength as a monolithic structure. It is also cheaper, and its perfectly smooth inner surface is a valuable aid to ventilation.

Colliery Work.—The application of the lining to underground main roads in collieries presents many advantages; to mention only one: as there are no projections, coal dust cannot collect.

By way of example this new system may be compared to two methods at present in use.

1. A lining made of brick arches with filling of mass concrete behind.

The block lining may be made and placed into position practically by unskilled labour; thus there would be a distinct saving as compared with masonry work.

Reinforced concrete offers greater resistance than masonry, so the lining may be made thinner, thus saving about 10 per cent. to 20 per cent. in excavation work.

Finally, the blocks can be placed without the use of centering.



Fig. 2. Plan.

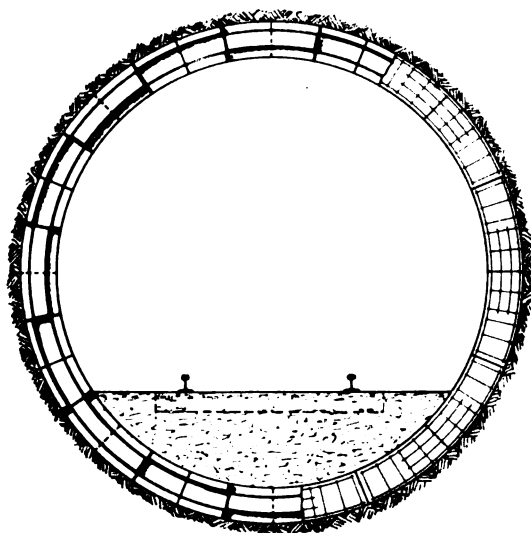
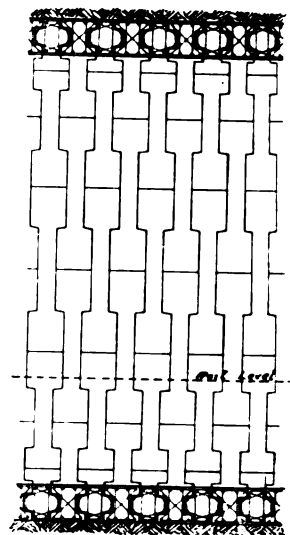


FIG. 3. Cross Section



Longitudinal Section.



FIG. 4. Demonstration Arch Erected at Willesden.
AS APPLIED TO TUNNEL WORK.

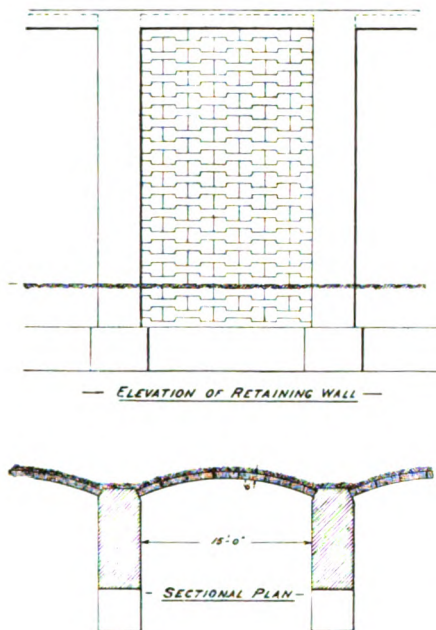


Fig. 5.

of blocks, thus saving centering and accelerating the progress of the work. Of course, the blocks may be made straight, thus forming any kind of reinforced concrete slab—viz., floors or walls.

For shore protection or the slopes of canals the blocks are easily laid out, and when once placed in position the interlocking would prevent the dislocation of any block.

For the manufacture of the blocks on a larger scale special machinery would be applied for the moulding operations and to ensure the greatest possible strength of the blocks. Their dimensions may be such that they can be conveniently handled by two men. They may be of such a size that they could be lifted by means of a light tackle.

Any further information or details will be furnished on application to Messrs. Peter Lind and Co., at Parliament Mansions, Victoria Street, Westminster.

2. Steel frame with a lining of concrete between.

Here the advantage of casting on the surface comes into account. By casting underground the strength of the lining will suffer through the unavoidable entrance of coal dust into the concrete whilst it is being mixed, and also owing to insufficient ramming.

Again, centering can be dispensed with, which is, of course, a saving.

The same advantages are claimed for mine shafts.

Generally.—By means of these interlocking blocks any kind of arching and plain slabs may be constructed in addition to tunnel work—as, for example, arched bridges, etc.

Fig. 5 shows how the blocks may be used for the construction of a retaining wall.

In this case they form horizontal arches between strong buttresses. These would be cast first and given proper time for setting; then, as the excavation proceeds, the blocks would be placed in position.

In this way no strutting is required.

Fig. 6 shows the application of the blocks for the construction of river arching.

The bottom and sides are to be cast in the usual way, but the cover is to be built up

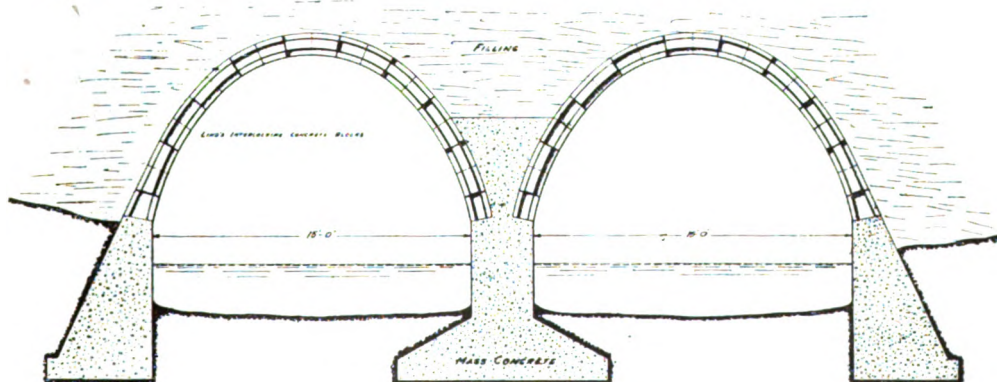


Fig. 6. Cross Section of River Arching.

CORRESPONDENCE.

Under this heading we invite correspondence.

To the Editor of CONCRETE AND CONSTRUCTIONAL ENGINEERING.

SIR,—Your editorial in July issue headed "Contractors and Reinforced Concrete" makes interesting reading to anyone like myself, who, during the past ten years in which I have been associated with reinforced concrete construction, has been brought to the realisation that much remains to be done to establish a more satisfactory state of affairs in general. Speaking of my individual experience and conviction, I feel sure that reinforced concrete construction, in order to be developed to its best, must be specialised in as far as the contracting and constructional side is concerned. There are many reasons for this, the following are some:—

1. Trained men are necessary. It is commonly claimed that any unskilled labour is suitable—this is but a half-truth, because this applies solely, and then only in part, to the labour which does the work of mixing and conveying the concrete, and I maintain that the other labour, bar bending, assemblage, placing in position of steel and concrete, the punning of concrete and the making and erection of scaffolding, staging, and casings, can only be done satisfactorily when men are employed who are used to this particular class of work.
2. In the perfect equipment for handling any job in the most speedy and economical manner an intelligent contractor or contractor's man can evolve gear and labour-saving appliances, the cost of which would be prohibitive for one small job, but if considered as stock plant the cost would be spread over many succeeding contracts and effect considerable saving in time, labour and cost.
3. One of the important considerations in reinforced concrete construction is time in construction. At one period, perhaps, very little carpenters' work is to be done; then for a short period a considerable number of carpenters could be employed. Men cannot be obtained for such short periods, and even if they could, their utility would be limited on account of their newness to the work. The solution, as far as I can see, is that of the expert specialist firm with sufficient work of this class always on hand, either in the shop or on works to enable them to place men as required.
4. Skilled contractors, managers, and foremen are also necessary, who themselves would ensure the honest claim of a contracting firm which aspires to reinforced concrete expert rank. An ordinary foreman would not be conversant with the principles of design and the technical training as applied to reinforced concrete, but in course of time and experience in carrying out work he becomes generally conversant with the guiding principles or features of construction. Nevertheless, it is imperative that a man technically trained and more perfectly educated should maintain supervisory touch with the work as representing the contractors.

I would like also to put this question: Is the ordinary "clerk of works," who is accustomed only to such duties in connection with other and older classes of construction— is he fitted, as a rule, for the somewhat autocratic position which in reinforced concrete construction he sometimes assumes? When a clerk of works is employed it would lead to much greater satisfaction if he were a specialist in this class of work. By this I mean that he should be not only what is understood as a "practical man," but he should be actually acquainted with the principles of design and so be capable of real judgment in contrast with the man who, through insufficient knowledge, safeguards himself behind a policy of perfection to such a degree which, besides being irksome and uncalled for, is often the cause of serious delay and expenditure.

Reinforced concrete design and reinforced concrete construction should form a perfect combination, and it is very essential that whoever has the control of any reinforced concrete constructional work should be capable of well balancing the necessities of design and the reasonable sufficiency and accuracy in construction.

5. To follow literally the cautious provisions and requirements as laid down by many designers and writers in construction would, I should say, be the likeliest way

of turning a man's hair grey in the shortest time on record. No doubt the reason for such very perfect warnings and instructions emanate from the misgivings which designers have as to the capability of possible constructors fully to appreciate the really necessary care, watchfulness and skill required to ensure reasonably sound construction. This is another point in favour of the plea for specialist skilled constructors, who, with the knowledge of experience could be depended upon to construct in a thoroughly workmanlike manner without endeavouring to do impossibilities, and on the other hand without committing errors which would be detrimental.

6. Another item of importance, often quite unprovided for, is the method by which the ultimate finished structure is obtained--this can affect the varied interests materially; the constructor as regards cost and time, the client also as regards cost and appearance, and the designer also on all these points. In many contracts methods of construction including the matter of surface finish and appearance are quite ignored until the work of construction actually takes place, with the frequent result that differences of opinion occur between the contractor, clerk of works, client and designer, which may possibly lead to serious trouble and disappointment, and on the contractor's side possibly serious loss.

7. Standardisation appears also to require some consideration. The standardisation of casing sizes and concrete scantlings, the sizes and lengths of steel reinforcement, design and finish, would all be in the interests of economy and speed. Cement also might be classified as regards its setting properties, and it seems desirable that some agreement should be come to as to the permissible relationship between quickness of setting and ultimate strength. The progress of many structures can be materially affected by the setting speed of the cement used.

8. In the matter of design, irrespective of actual strength conditions, especially in structures of reinforced concrete in its purest forms, such as bridges and water towers, etc., it appears that to be able to design according to the particular needs of any scheme it is very necessary that one should have a sound experience in actual construction in order that those needs may be properly considered. For instance, where time and cost are extremely important factors, the introduction of projections and mouldings such as break the regularity of the cased surfaces can make a very appreciable difference in these items.

9. A reference to the various publications and textbooks on Reinforced Concrete will show how varying are the laws laid down, some of these dealing with operations very vital to progress and cost.

To anyone who is regularly in touch with constructional problems it cannot be but clear that academic rules are much too narrowing and cannot be sufficiently comprehensive except to serve as a very proper guide to the novice and a protection against errors of inexperience.

The successful prosecution of Reinforced Concrete practice must rest on the results of specialised and continuous efforts and experiences.

I am, yours faithfully,

V. D. LAKE.

Passmore House, Topsham, Devon.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Westminster Technical Institute.—The 1915-16 Session of Evening Technical Classes was started at the end of last month. Classes are held in the different branches of Architecture and Construction and of Civil Engineering, and are divided up into junior and senior courses. The classes in Reinforced Concrete Design in the Engineering Section are held every Wednesday from 6.30 to 8.45 p.m., whilst the Architectural Class in the same subject is held every Friday from 7.15 to 9.30.

Full particulars of all the classes will be sent on application at the Institute at Vincent Square, Rochester Row, W.C.

University of London—The new syllabus of the School of Architecture for 1915-16 is now to hand. Structural Engineering Classes for Students will be held as follows, commencing October 4th: First term, Mondays and Thursdays at 4 p.m., Tuesdays at 5. Laboratory work, second term, Mondays, 10-5. The Classes will be held at University College, Gower Street, W.C. Full copies of prospectus, etc., may be obtained on application to the Secretary.

The Manchester Municipal School of Technology—The prospectus for the winter session is now obtainable on application to the Principal, School of Technology, Manchester. Evening classes will be held in every branch of engineering work, and are arranged in progressive courses.

Zeppelin Raids—Police Warning.—Through the official Press Bureau the following police warning to the public in the metropolitan area has been issued:—

"In view of the possibility of further attacks by hostile aircraft, the Commissioner of Police deems it advisable to call attention to the public warning published on June 28th last, recommending residents to remain under cover, and advising them, for dealing with incendiary fires, to keep a supply of water and sand readily available. The opportunity was taken of suggesting that chemical liquid fire extinguishers should not be purchased without a written guarantee that they comply with the specification of the Board of Trade, Office of Works, Metropolitan Police, or some such approved fire prevention committee.

"Many dry powder fire extinguishers are much advertised. The Commissioner warns the public, as the result of experiments made at his instance by a competent committee of experts, that no reliance can be placed upon such appliances for effectively controlling fires such as are likely to be caused by bombs, explosive or incendiary. The Commissioner is advised as the result of these experiments that the provision, and prompt and intelligent use, of water or of sand, or of both, in dealing with such outbreaks of fire is the best, simplest, and most economical safeguard.

"E. R. HENRY, Commissioner of Police of the Metropolis."

"September 18th.

Reinforced Aeroplanes—The principle of reinforced concrete as applied to roads is now apparently being used in connection with the manufacture of the latest and most up-to-date type of German aeroplane. Wings composed of "slabs of synthetic resin, reinforced with wire mesh, and metallic rods made of alloy," to quote a daily paper of wide circulation, have been tested and found to be not only of great tensile strength, but practically "invisible." Reinforcement of this type is a novelty, which will probably appeal to the firms who manufacture reinforcing fabric.

Cheaper Road Making.—The Borough Engineer of Kensington has prepared a scheme to repair roads economically. Much road improvement is necessary, but municipalities will not be able now to secure loans for costly repaving schemes. The Borough Engineer some time ago experimented with a concrete carpet over the macadamised roads, and was delighted to find that it served its purpose admirably. The carpet not only spread the weight of the heavy modern motor vehicle fairly evenly over the foundation of the road, but also did much to absorb shock. Now this process is to be adapted to certain wooden roads which are in bad condition. He proposes to deal with an experimental length of roadway by removing the wood and converting the foundations into a concrete road. The existing foundation will remain, but will be reinforced by a layer of bituminous concrete, over which a wearing surface carpet of concrete will be placed. It is computed that such a reinforced road will last as long as a completely new road, but the cost will be only one-third of a new road.

Concrete for the Keel of a Sailing Craft.—Concrete has replaced lead as a material for the keel of a small sailing craft in Pennsylvania. The concrete keelsom weighs 2,200 lb., and was fitted on the auxiliary yawl of a boat 33½ ft. width and 10½ ft. beam. The cost was £5 as against approximately £45 if lead had been used. It is stated that so far the experiment has proved successful. The vessel has seen hard service and has been aground several times, whilst the keelsom is apparently still in good condition.

Reinforced Concrete Roads at Chiseldon, Wilts.—An interesting experiment in road construction has been made at Chiseldon Camp for the War Department, about five miles from Swindon, where some concrete roads have been constructed, and others are in course of construction in different sections. Altogether there will be about two miles of these roads. The surface formation of the roads is concrete, 6 in.



thick, reinforced with B.R.C. reinforcement, made by the British Reinforced Concrete Engineering Co., Ltd., Manchester, who are constructing the roads. The roads comprise different sections, one being 700 ft. long and 20 ft. wide. It was originally intended that the concrete should be covered with asphalt or similar material, but eventually it was decided to leave the concrete surface as it was. Another road was 15 ft. in width. The concrete mix is 5½ to 1. The reinforcement takes the form of electrically welded steel wire placed about 2 in. from the underside of the concrete.

Cement as Anti-Sand Blast.—The use of cement as an anti-sand blast is deserving the attention of all railway engineers. When locomotives stand under steel bridges or viaducts, the blast from the smoke shaft throws out gases and numerous small particles of ashes and coal, causing serious injury in course of time. In such cases cement has been successfully used, and now it is common practice on American railways to cover

The BRITISH STEEL PILING CO



A PLANT FOR EVERY PILE.

The above photograph shows how easily our McKiernan-Terry Patent Double-Acting Steam or Air Hammers can be adapted to work cheaply under diverse circumstances. Here a No. 5 size is driving timber piles for Groynes at Hythe. By suspending it from an aerial cable, work was continued irrespective of the tides, which submerged the work. These Hammers can be used under water—at an angle—inverted—anyhow—and give satisfaction under all conditions.

A PILE FOR EVERY PURPOSE.

Weights of Piling on sale or hire :—

"SIMPLEX"	-	22 to 27 lbs. per sq. ft.
"UNIVERSAL JOIST"	-	from 43 lbs. "

DOCK HOUSE, Billiter Street, LONDON, E.C.

the bottoms of bridges with reinforced concrete or cement mortar, which has been found to resist the attack of gases and also the blast of particles coming from the locomotives.—*Page's Weekly.*

Polish on Concrete Surfaces.—The amount of polish which can be given to the surface of concrete depends upon the density of the mixture and the nature of the aggregate used. After the surface has been smoothed down on a rubbing bed, or by the use of rotary rubbing stones, as applied in terrazzo floor work, the procedure is somewhat similar to that used in polishing granite or marble. The aggregate exposed on the surface by the rubbing process takes the polish, the appearance of the surface being dependent upon the percentage of aggregate exposed.—*Canadian Engineer.*

Reinforced Concrete Lighthouse.—The light station proposed for Navassa Island, in the Caribbean Sea, will be constructed by the United States Lighthouse Service, and will be on the southern side of the island. The tower will be cylindrical, of reinforced concrete, supporting a cylindrical helical bar lantern with a focal plane about 152 ft. above its base and 402 ft. above sea level. The illuminating apparatus will consist of two four-sided fourth-order lenses, mounted side by side, with their sides parallel with each other, and carried on a mercury float. The two parallel beams of light from the lenses combine to form a single beam of about 56,000 candles. The characteristic will be a double white flash every thirty seconds. The range, due to the height of the light, will be about twenty-seven nautical miles.

Reinforced Concrete Pipes.—The City Engineer of Port Elizabeth, Mr. A. S. Butterworth, will be pleased if makers of plant for the manufacture of reinforced concrete pipes would communicate particulars to him, c/o Messrs. Davis and Soper, Ltd., 54, St. Mary Axe, E.C.

Mr. Butterworth is anticipating an early visit to England in connection with the immediate execution of the scheme, estimated to cost about £500,000, for the completion of the sewerage of Port Elizabeth.

If engineers who have now had experience over a number of years of the wearing properties of concrete pipes would be good enough to communicate, Mr. Butterworth would be pleased to have the privilege while on this side of calling upon them.

PROPOSED WORKS.

Sedgfield.—The Urban District Council have approved plans for a picture hall at Fishburn to be constructed of concrete.

Conway.—The tender of P. W. MacLellan, Glasgow, amounting to £4,625, has been accepted by the Conway and Colwyn Bay Water Board for concrete foundations and erection of a girder bridge at Maenan.

China.—It is reported by Mr. G. A. Combe, acting British Consul at Chefoo, China, that the contract for the breakwater at Chefoo has been awarded to the Netherlands Harbour Works Co., whose tender was £365,300. The works include a breakwater and pier.

Lowestoft.—The Local Government Board has consented to the Town Council borrowing £7,500 for sea defence works.

Sheffield.—Tenders have been invited during the month by the Sheffield Corporation Electric Supply Department for concrete foundations for four new boilers.

Lisbon.—A law has been passed authorising the Government to raise a loan of 5,000,000 escudos for the execution of port improvement works, including the construction of two breakwaters.

Cape Town.—The City Engineer has prepared a scheme for the disposal of sewage in Cape Town suburbs estimated to cost £489,000. This includes an extension of the sea outfall.

Grain Elevator at Manchester.—The new grain elevator of the Ship Canal Co. at the east end of No. 9 Dock, which is now completed, has a holding capacity of 40,000 tons, and is capable of handling grain at the rate of 1,200 tons per hour. It is a reinforced concrete building with steel window frames and doors. The structure is 160 ft. high, 295 ft. long and 165 ft. wide.

PUBLICATIONS RECEIVED.

Messrs. McGraw Hill Publishing Co., Ltd., have sent us their 1915-16 List of Books. This is a useful list for reference, and includes a wide range of technical and scientific books. The list may be obtained on application to the above company at 6 and 8, Bouverie Street, Fleet Street, E.C.

CONTRACTS ACCEPTED.

The Unit Reinforcement Construction Co., Ltd., 49, King Street, Manchester, have recently secured the following contracts:—

Large extensions to stores in Reinforced Concrete for Messrs. J. Irwin and Sons, Liverpool. Messrs. Medcalf and Medcalf, Architects.

Reinforced concrete floors, roof, etc., to new schools, Rochdale. Mr. P. W. Hathaway, Corporation Architect.

Reinforced gallery, floors and all constructional work for large new picture house at Wigan. Mr. W. Ellis, Architect.

TRADE NOTICES.

Loop Road Reinforcement.—An interesting little booklet has just been issued by the Loop Road Reinforcement Co., dealing with a new form of reinforcement for roads.

It is claimed for the loop road reinforcement system that it has a high yield point and a mechanical bond, both essential qualities in reinforced concrete road foundations.

This loop reinforcement is supplied in convenient lengths for threading together on the site and can be applied in various positions in the concrete.

The following are some of the advantages claimed for this method of reinforcement: Provision for shear due to heavy concentrated loads. Distribution of heavy loads over a large area of ground surface and consequent reduction of tendency to local subsidence. Resistance to damage due to rolling out action of advancing load. Reinforcement easily handled, adapted to any width of road, and can be laid in half sections. Considerable economy over plain concrete slabs of greater thickness—that is to say, a 6-in. loop reinforced concrete road bed is stronger and cheaper than 9 in. of solid concrete. No liability of reinforcement to become displaced when concrete is put in position. Can be efficiently reinstated after trench opening.

For further particulars, prices, etc., apply to the Loop Road Reinforcement Co., 663, Caxton House, Westminster, S.W.



THE **VICTORIA** CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.



Photograph by Ernest Milner, London, S.W.

Exterior View of Completed Building.

THE ISLAND SITE HOTEL.

(For description see page 515.)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X. No. 11.

LONDON, NOVEMBER, 1915.

EDITORIAL NOTES.

THE INFLUENCE OF THE WAR.

THE influence of the war is very far-reaching, and there is not a single individual who is not affected in some way or other, either directly or indirectly, as regards his business. Apart from the personal physical suffering and mental anxiety which has been caused, the war has considerably altered the outlook of many firms, bringing prosperity to some and ruination to others, while in many instances the nature of the business has been entirely altered by the proprietors, who have realised that their resources could be beneficially applied to the needs of the country for war purposes during a time when there is little or no demand for the goods ordinarily produced.

The building industry has probably been affected as much as any other, as clients who contemplated extensive or speculative building before the outbreak of war have abandoned their schemes, while municipal expenditure has been reduced to a minimum; and thus the ordinary sources of work have practically disappeared for the time being. It does not follow, however, that building work is scarce because ordinary work is nearly suspended, and, in fact, there is a great amount of building proceeding in this country, but it is of a different class. There is a shortage of labour, and a difficulty in obtaining and transporting materials, and, despite these two factors, work is carried out speedily when well organised, if the structure is for war purposes, as are nearly all those buildings now in hand. Many firms who have executed sections of building work in the past are now engaged upon the supply of munitions, and thus the available sub-contractors are considerably reduced, while many large building contractors are so busy with large Government orders that they are unable to undertake any further work.

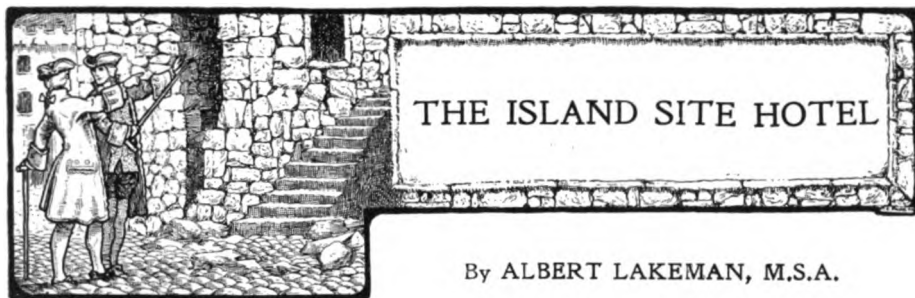
The architect and the engineer are thus faced with many new difficulties in securing the speedy execution of any work which they may have in hand, and it behoves them carefully to consider all the aspects of the case when designing the structure. It is of great importance to remember that certain materials can be obtained more easily than others, and it is necessary to make a judicious selection to avoid delay in carrying out the work owing to one of the essential elements in the construction not being available.

It is here that reinforced concrete will be found advantageous as compared with ordinary constructional steel, and the war has done much to show up the possibilities of the former material. There appears to be little or no difficulty in obtaining ordinary round bars for reinforcement, and these are

easily transported, while rolled steel sections are difficult to obtain, and built up members involve work in the shops which takes time and diverts labour which might be more profitably employed in the manufacture of shells and machines of war. Under proper supervision the labour required for reinforced concrete work is practically unskilled, and its employment on the building of war factories is an example of the proper application of our resources, as such men could be of little use to the country in any other sphere apart from the Army itself. There are also other points to be considered in dealing with a comparison of steel and reinforced concrete, and one of these is the question of economy. Owing to certain stock steel sections not being available at the present time the designer is compelled to adopt the next largest size that can be obtained, and, in consequence, more steel is employed than is theoretically necessary. This does not hold with reinforced concrete, as the designer can vary the number of bars used, and the fact that any particular diameter bar is not at hand will simply mean less larger bars or more smaller bars, each providing the requisite area of steel and no more. Some designers will raise the point that the concrete structure must be allowed to stand for a period after execution before being used, but as a set-off against this it must be remembered that the bars can be obtained and the work commenced immediately, whereas the erection of the steel cannot be commenced for some weeks, even if the sections required are being rolled at the mills, owing to the preparatory work that has to be executed in forming connections and caps and bases. No plastering is necessary with reinforced concrete factory buildings, and yet a good surface is obtained; while with properly designed reinforced concrete floors, cracks which so commonly occur with steel fillers and concrete are avoided, and this is of great importance in some of our war structures.

It will be obvious that in munition factories and war buildings generally architectural effect is of secondary consideration and construction is all-important, and the advantages of reinforced concrete therefore affect the very essence of building at the present time. Architects and engineers generally deserve credit for the way in which they have adapted themselves to war conditions, one class of practice having been entirely superseded by another, and the second calling for strenuous work during long hours and causing much anxiety on account of the difficulties of labour and materials; and it is some consolation to feel that the endeavours made in the past to obtain the just recognition of reinforced concrete are now bearing fruit. At the same time, the difficulties of these architects and engineers are being lessened by the extensive use of this material.

It is manifestly impossible to give any details of the actual war structures in which reinforced concrete has been employed, and it would not be advisable to publish such information even if it were available; but the majority of our readers will have had sufficient experience in building work since the commencement of the war to realise that reinforced concrete as a constructional material has proved its worth during these exceptional times, and the influence of the war has been such that it will continue to be the most extensively used material for building purposes even when the happy days of peace return once more.



In the following article we are describing what is mainly a steel frame construction; reinforced concrete was used in the staircase construction and for some of the floors, and there is also an interesting application of concrete blocks.—ED.

THE design and construction of large hotel buildings are always of interest on account of the numerous problems encountered which vary in every building erected, while the requirements of comfort, service, and architectural treatment impose limitations on the constructional engineer which call for ingenuity in dealing with these problems. This particular building, named the Regent Palace Hotel, is said to be the largest hotel in Europe, there being over one thousand bedrooms on the upper floors, and this fact, together with the irregular shape of the site, renders a study of the design and methods adopted of value to architects and engineers.

The architects are Mr. Henry Tanner, F.R.I.B.A., and Mr. F. J. Wills (late Mr. W. J. Ancell), and the clients for whom the building was erected are The Strand Hotel, Ltd.

General Description.—The site, situated close to Piccadilly Circus, has frontages of about 275 ft. to Glasshouse Street and about 300 ft. to Sherwood Street, in addition to smaller frontages to Brewer Street and Air Street, and the irregular shape can be seen upon referring to the ground-floor plan illustrated in Fig. 1. The principal entrance is arranged at the apex formed by the Glasshouse and Sherwood Street frontages, thus being close to Piccadilly Circus, and the main axis of the building is given by bisecting the angle given by the convergence of these two fronts. The principal rooms are arranged with their centre lines on this axis, thus giving a fine vista from the entrance lounge, which is of importance in a building of this nature, and, in addition, the best possible arrangement is obtained. The entrance lounge is circular on plan, with a diameter of about 36 ft., and from this the visitor passes into a large hall, around which are grouped the offices, staircase, and lifts. From this hall three sets of double doors lead to the winter garden, which extends across the building from Glasshouse Street to Sherwood Street, a distance of over 130 ft. The main central portion of the winter garden is octagonal on plan, with a width of 72 ft. between two opposite faces, and this is roofed with a glass dome, 40 ft. diameter, having a height of about 18 ft. to the springing, thus providing a well-lighted apartment. Immediately behind the winter garden is the coffee and dining-room, which has a maximum length of 114 ft. and a maximum width of 100 ft. This is a fine, large room with unbroken floor

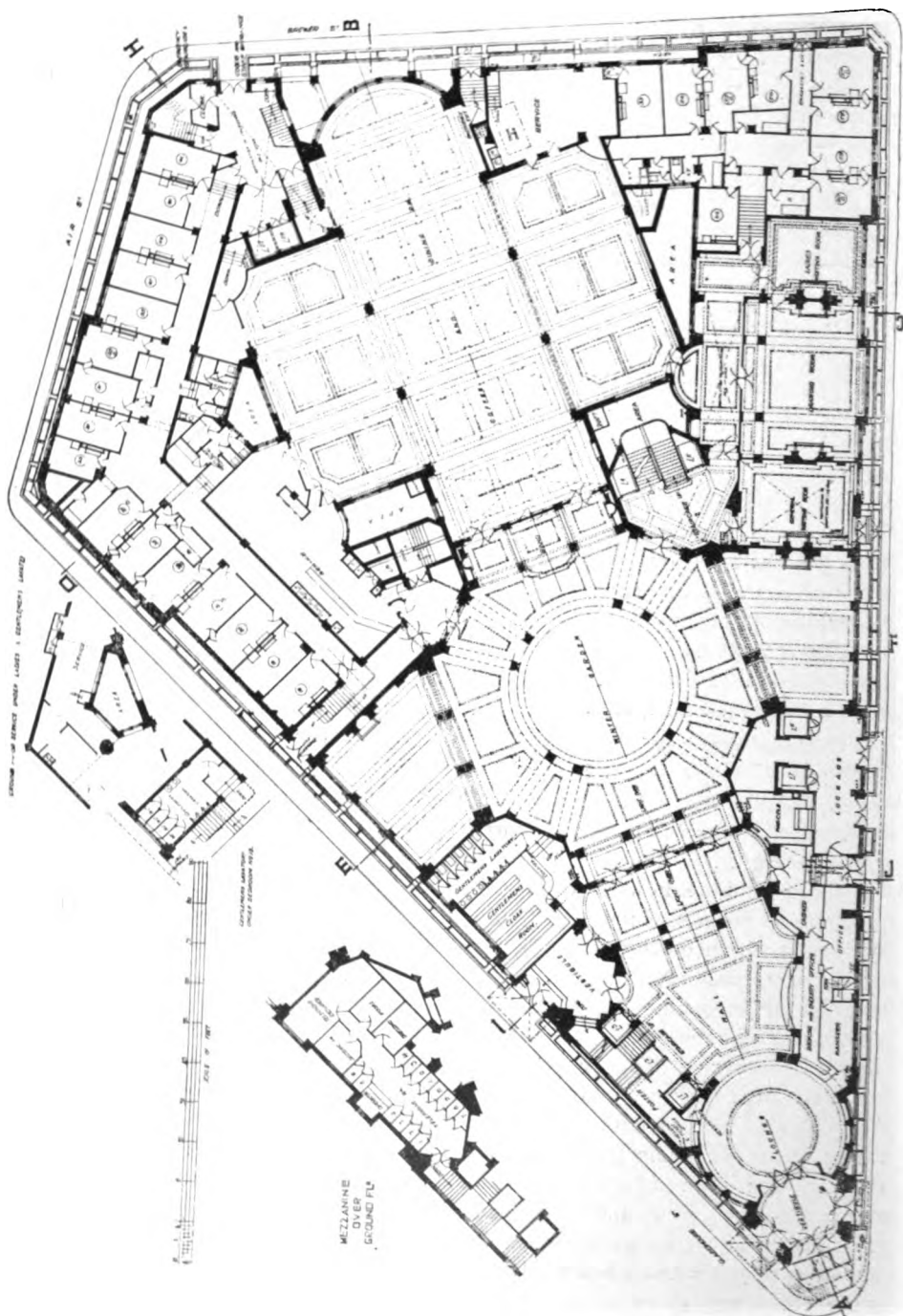


FIG. 1. Ground Floor Plan.
Top and Side Views.

space, with the exception of four small supporting members, and service rooms are arranged on two sides to ensure quick and efficient service. A large drawing-room and writing-room are also provided, and a separate entrance and vestibule are arranged on the Glasshouse Street side in connection with the grill-rooms, which are situated in the basement.

The luggage is taken to a separate entrance in Sherwood Street, where a special luggage room is arranged, from which two large lifts rise up to the various floors. A goods and staff entrance is arranged in Brewer Street, and the remainder of the ground floor is taken up with cloakrooms, lavatories, staircases, and a certain number of private rooms. The basement is devoted to grill-rooms, billiard-rooms, toilet saloons, kitchens, staff rooms, and all the various departments required in running an establishment of this kind, while the whole of the upper floors are given up to bedrooms, each of which is provided with a lavatory basin with hot and cold water.

The exterior of the building is treated in a very successful manner, as will be seen by the elevation in *Fig. 2*, the facing employed being a glazed terracotta known as "Marmo," and the roofs are covered with green slates. A section of the building is illustrated in *Fig. 4*, and the height from the basement-floor level to the roof is about 126 ft., there being nine floors above the ground floor, while, in addition to this, there are various ducts and passages below the basement, some of which are 9 ft. high, for pipes and in connection with the air supply for ventilation purposes.

Construction. — The structure itself is designed and executed as a steel-framed building; the steel, which weighs about 6,000 tons, was practically all erected as a skeleton before the remainder of the work was executed. The height of the building being great, special provision was made for the lateral bracing of the external stanchions, as described hereafter, and some interesting details are to be found in the steelwork.

Foundations. — The foundations are carried down to varying levels according to the nature of the underground work in the immediate vicinity of any particular stanchion, and, generally speaking, grillages were adopted throughout. Many of the foundations under the stanchions are comparatively small, but the largest type covers an area of 9 ft. by 7 ft. 6 in., this occurring under some important stanchions in the centre of the building. The steel employed in this type is arranged in two layers, the lower consisting of nine 15-in. by 6-in. rolled steel joists and the upper layer being formed with two compound girders placed closed together, each built up with four 15-in. by 4-in. channels and four 16-in. by $\frac{3}{4}$ -in. flange plates with channel and angle stiffeners immediately under the stanchion and at intermediate points in the length. The rivets in the top flange under the stanchion base are countersunk, and the base plate is bolted to the girders with $\frac{7}{8}$ -in. diameter bolts. Each joist in the lower layer is connected to the compound with two bolts, and the whole is encased in concrete finished 10 ft. below the basement-floor level. Many of the smaller grillages consisted of a single layer of beams only, the number employed varying from two to seven, and these were bolted together with short pieces of channel placed between the webs. The greater number of the foundations to

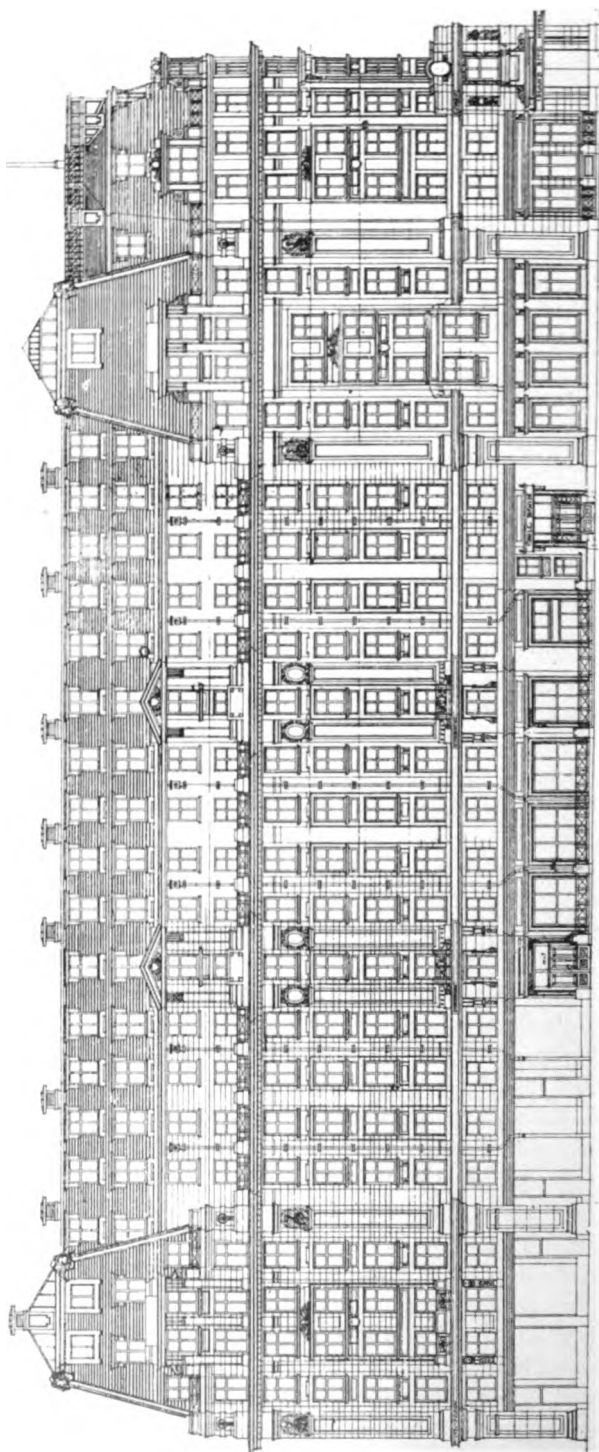


FIG. 2. · Elevation to Glasshouse Street.
ISLAND SITE HOTEL.

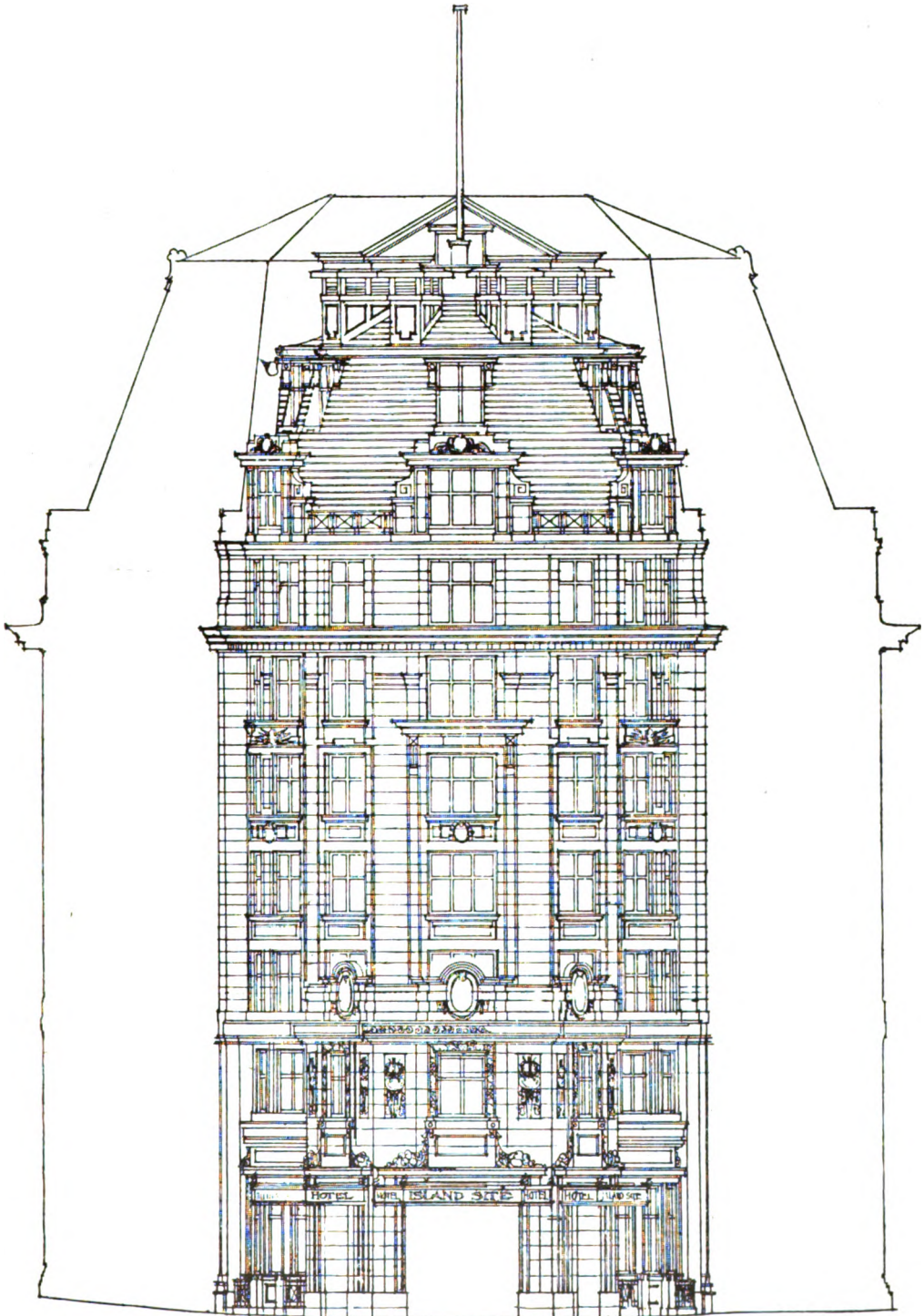


Fig. 3. Elevation of Apex.
ISLAND SITE HOTEL.

the stanchions were carried down 9 ft. below the basement-floor level, and the maximum depth was 13 ft.

Stanchions.—The lay-out of the stanchions is of necessity somewhat irregular owing to the shape of the building, but the spacing is, generally speaking, at about 20-ft. centres. In setting out the stanchion plan the building lines were carefully laid down and the angles figured, and the principal axis line above-mentioned was drawn in together with the centre lines of the corridors on the upper floors, these corridors running right round the building, and the whole of the figuring was then given in connection with these lines. This involved considerable care and accuracy in the initial work, but once the general lay-out was accomplished the work could be designed and erected without risk of serious errors.

There were practically four types of stanchions employed, viz.: (1) the ordinary rolled steel joist; (2) the ordinary compound section composed of rolled steel joists and plates; (3) a type of twin compound; and (4) an H-section built up with three rolled steel sections, one of which is placed at right angles to the other two.

The first-mentioned type was employed for secondary stanchions and in the roofwork where the setting back necessitated using vertical supports which extended through the height of one floor only and no considerable load had to be supported. The size of the second type varied considerably, but a typical example shows a section built up of two 20-in. by $7\frac{1}{2}$ -in. R.S.J. and four 20-in. by $\frac{3}{4}$ -in. flange plates. This stanchion has a base plate 4 ft. 2 in. by 2 ft. 8 in. and gusset plates 1 in. thick and 3 ft. 3 in. high, with $3\frac{1}{2}$ -in. by $3\frac{1}{2}$ -in. by $\frac{1}{2}$ -in. angles along the outer edges to assist against buckling. The two rolled steel joists were connected by 10-in. by $3\frac{1}{2}$ -in. channels placed at 4 ft. intervals between the webs, these channels being riveted with $\frac{3}{4}$ -in. tack rivets to one joist before building up, and 1-in. through bolts were afterwards inserted to draw the webs tightly up to the stiffening channels. The twin compound stanchions were built up with four rolled steel joists, each 12 in. by 6 in., and eight flange plates, each 14 in. by $\frac{3}{4}$ in. The section actually consists of two compound stanchions, each of which is built up with two joists and four plates, and these two parts are connected at the cap and base and at intervals in the length by 18 in. by $\frac{1}{2}$ in. steel plates riveted across the flanges, giving an overall size of 2 ft. $4\frac{1}{2}$ in. by 15 in. In some cases twelve flange plates were employed, and such sections were capable of carrying a considerable load and were a convenient shape to be employed in their particular positions. The fourth type was built up with three 12-in. by 6-in. R.S.J., one of which was riveted between the webs of the other two and at right angles to them, and in addition eight 6-in. by $\frac{3}{4}$ -in. flange plates were used, these being attached to the outer joists only. Connecting-plates, 9 in. wide, were used to tie these two outer joists together at intervals.

Girders.—In the girder work some interesting details are to be found in the larger sections which act as the main beams. The floors are constructed with small filler joists varying from 4 in. by $1\frac{3}{4}$ in. to 8 in. by 6 in., and main beams either carried directly by stanchions or supported by large compound

girders, which are themselves carried by the stanchions. In many instances two compound girders side by side are used, and these are carried by the twin compound stanchions previously described.

These girders have a span of about 43 ft., and each compound is built up distinctly with large angle stiffeners on both sides of the web, and they were

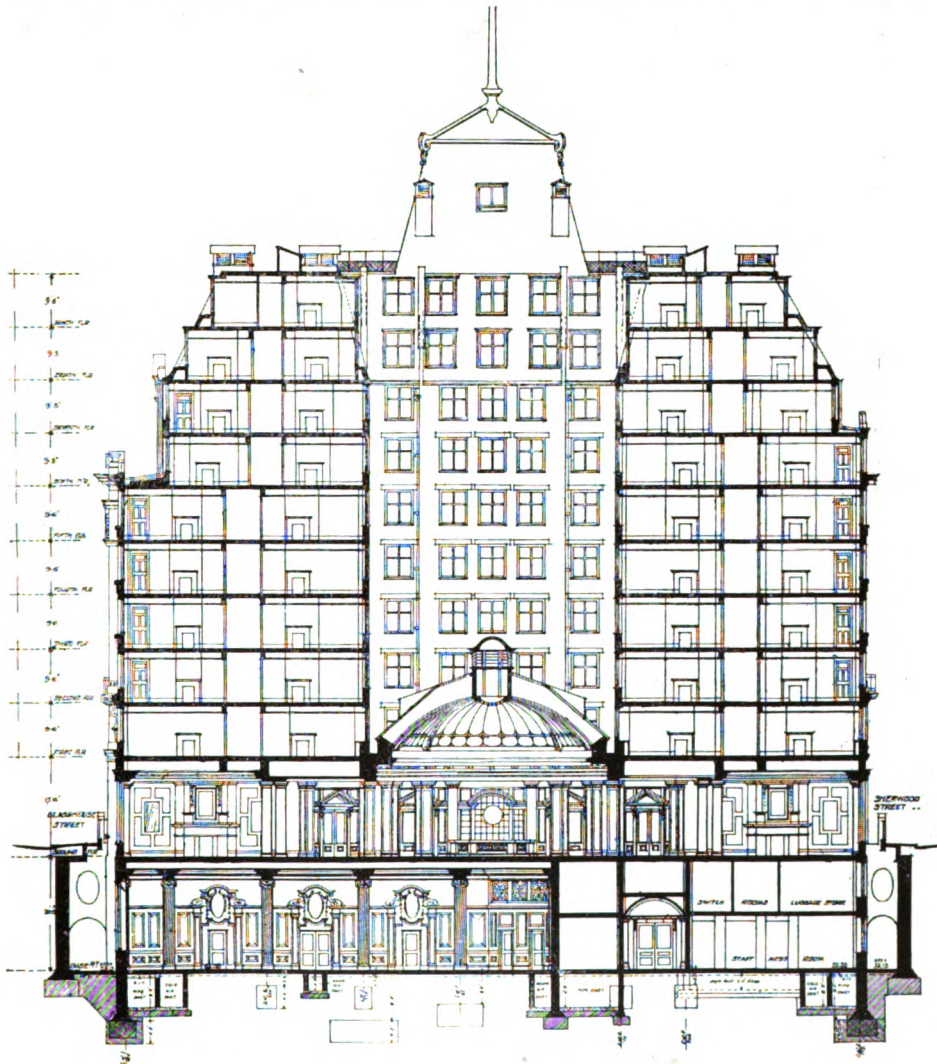


Fig. 4. Section.
ISLAND SITE HOTEL.

afterwards connected to form one large twin girder by riveting a plate between the webs. This was accomplished by taking a plate with an angle at either end and riveting this by means of the angle to the web of one girder, in such a position that upon the girder being placed together the other angle

came against the second web. Hand holes, 4-in. diameter, were formed in the second web for use when bolting up in position, thus allowing the second connection to be made. This method allowed of easier transport to the works and easier erection, as only one-half of the weight had to be raised at one time, and in addition the webs were stiffened more thoroughly than is usual with compound sections of the box type. An example of a large plate girder is one having a span of 42 ft. and a total depth of 5 ft. 9 in., this being built up with two $\frac{3}{4}$ -in. web plates, four 8-in. by 8-in. by $\frac{3}{4}$ -in. angles, and sixteen 18-in. by $\frac{3}{4}$ -in. flange plates at the centre, reduced to four plates at the ends. Angle stiffeners, singly and in pairs, are used throughout the length, and the rivets used generally are $1\frac{1}{8}$ -in. diameter. Many other large girders were employed of the compound type, some of single web and some of double web section, but the two described above are probably the most interesting.

Reinforced Concrete.—A certain amount of reinforced concrete was used in the staircases and small floor slabs, but, generally speaking, the material was not extensively used.

Bracing of Steelwork.—Great care was taken to brace the steelwork in all directions, and this was accomplished by using raking rolled steel joists, which were connected to the stanchions in the external walls at their upper ends and carried back to the next line of stanchions, each line of bracing extending through two or more floors. The size employed varied from 8-in. by 6-in. R.S.J. on the lower floors to $3\frac{1}{2}$ -in. by $3\frac{1}{2}$ -in. by $\frac{1}{2}$ -in. angles on the upper floors, and the bracing was arranged to come in the partition walls, thus avoiding obstruction or loss of floor space. The roof steelwork was also thoroughly braced to compensate for the lack of continuity in the vertical members, these latter being in many cases only one storey high and being carried at their lower ends by main girders. The whole of the steelwork details indicate very careful design, and the structure is an excellent example of modern steel construction.

Concrete Block Construction.—An interesting instance of the use of concrete blocks is provided in the ventilation work, there being over 50,000 duct blocks of this material used in the building. These blocks were made on a "Winget" block-making machine and were 12 in. by $9\frac{1}{2}$ in. by 9 in. and 13 in. by $8\frac{1}{2}$ in. by 9 in., with 10-in. by $6\frac{1}{2}$ -in. core holes for single flues and 17 in. by 12 in. by 9 in. with two core holes with a division web 1 in. thick for the double flues. All internal corners are round to $1\frac{1}{4}$ -in. radius, and no rendering was required on the inside owing to the perfectly smooth surface obtained. All the blocks were made with a clinker aggregate, and special bevelled members were provided for gathering over where required.

Ventilation, Heating, and Lighting.—The ventilating, heating, and lighting systems have been carefully designed in accordance with the best modern practice, and an idea of the magnitude of this section of the work can be gathered from the fact that there are some 300 miles of pipes, 15 miles of air ducts, and over 180 miles of cable. The ventilation is on what is known as the "balanced system," and the air is washed before being sent into the building, with a battery of warm pipes for heating in the winter and a

refrigerator for cooling in the summer. The whole of the heating, ventilation, and hot-water supply systems can be controlled by the engineer from his office in the building, and the entire plant is so designed that any repairs can be

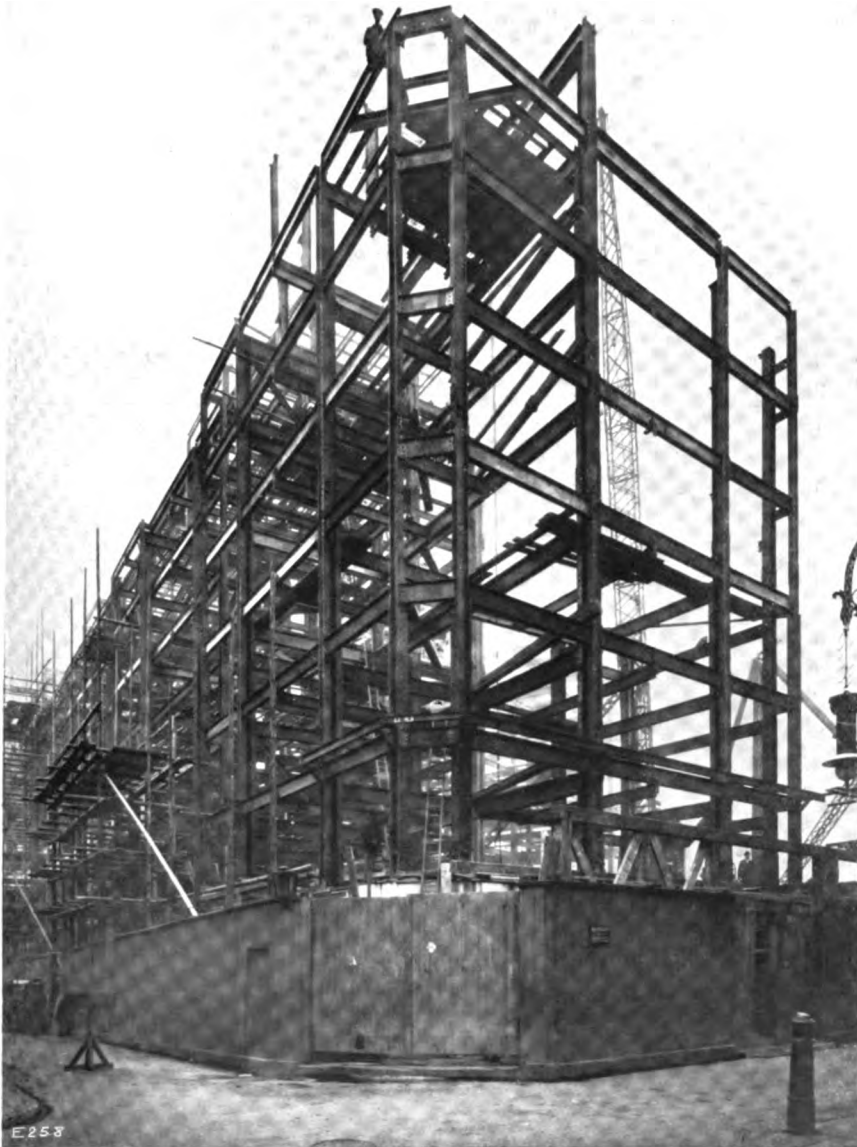


Fig. 5. Showing Building in Course of Construction.
ISLAND SITE HOTEL.

effected without a workman being seen by visitors. There are thirty-two lifts in the hotel, and these are all designed and arranged on a system which ensures

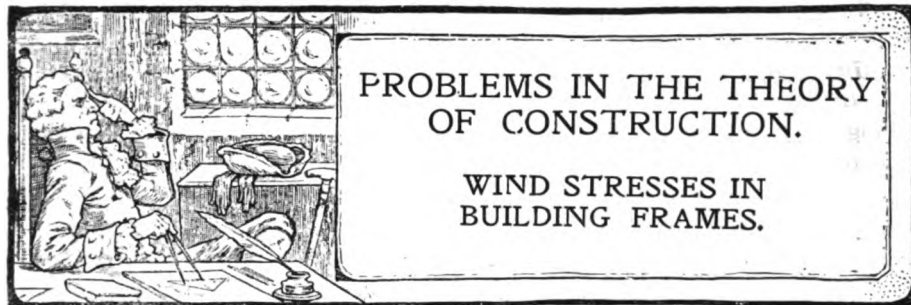
prompt attention while avoiding the possibility of two or three lifts going to the same floor to answer a call.

Generally.— The general contractors were Messrs. J. Mowlem and Co., Ltd., of Grosvenor Wharf, Westminster, and Mr. A. H. Barker was the consulting engineer for the ventilating and heating in conjunction with Mr. Cushion, of Messrs. J. Lyons and Co. The consulting engineers for the steel construction were Messrs. Reade, Jackson and Parry, of Bedford Row, and this work was executed by Messrs. Dorman, Long and Co. The concrete blocks were moulded on machines supplied by Winget, Ltd., 25, Victoria Street, Westminster, to Messrs. A. and S. Wheeler, 20, Abchurch Lane, E.C., who were responsible for the concrete block work, and the ventilating, heating, and hot-water systems were carried out by Messrs. J. Jeffreys and Co., Ltd. The Bovis patent reversible window fittings were adapted to all the windows with the exception of the metal casements, and the lift engineers were Messrs. Waycod and Co., Ltd.



Photograph by Ernest Milner, London, S.W.

Fig. 6. Interior View showing the Palm Court.
ISLAND SITE HOTEL.



By EWART S. ANDREWS, B.Sc.Eng.

The question of wind stresses is one of considerable importance in building construction, and the following article should prove of use and interest to all engaged in the designing of buildings.—ED.

THE stresses due to wind forces upon a rigid building frame cannot be found accurately except for the very simplest cases, and even then the analysis is very complicated.

In an article in the *Engineering News* of March 13th, 1913, Mr. R. Fleming explains three approximate methods which are used in practice, and illustrates them with reference to a numerical example. In the present article we will explain his third method at greater length, adopting the same numerical example.

The method is known as the "continuous portal method," and involves the following assumptions:—

(1) That the vertical or direct forces in each column are proportional to the distance of the column from the centroid of the group.

(2) That the points of contraflexure of each column are midway between the floor levels.

(3) That the horizontal or shear force in each column at a given level is equal.

Fig. 1 shows the frame to be considered, together with the wind forces (in thousands of pounds or kips) acting at each floor level. These forces are shown on the right-hand side.

Notation.—We will adopt the following notation for the columns and beams.

$A_{3,4}$ represents the column between

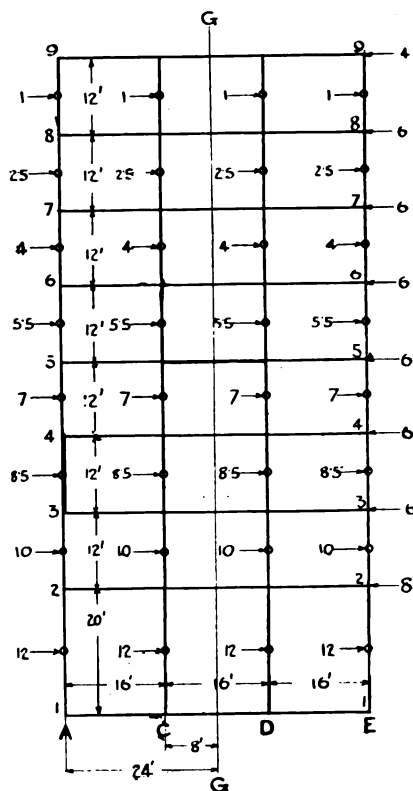


FIG. 1.

the points 3 and 4 on the left-hand outside column; this is indicated by a double line in the figure.

CD_5 represents the centre beam at the level 5, and is also indicated by a double line.

Vertical Forces in Columns.—Assuming, as is almost certain to be the case, that the outside and inside columns are respectively of the same section, the centroid line of the columns will be GG . Then the forces in the outside columns will be three times those in the inside ones because they are three times as far away from the line GG .

Therefore, taking moments about the intersection of GG and the horizontal line through the points of contraflexure, we have for the top columns :

$$E_{89} \times 24 + D_{89} \times 8 - C_{89} \times 8 - A_{89} \times 24 = 4 \times 6 = 24$$

$$\text{but } D_{89} = -C_{89}$$

$$\text{and } E_{89} = -A_{89} = 3D_{89}$$

$$72D_{89} + 8D_{89} + 8D_{89} + 72D_{89} = 160D_{89} = 24$$

$$D_{89} = \frac{24}{160} = .15 \text{ (tension)}$$

$$\therefore E_{89} = 3 \times .15 = .45 \text{ (tension)}$$

$$C_{89} = .15 \text{ (compression)}$$

$$A_{89} = .45 \text{ (compression)}$$

Similarly, at the line through the points of contraflexure of the columns 78 we shall have :

$$160D_{78} = 4 \times 18 + 6 \times 6 = 108$$

$$D_{78} = \frac{108}{160} = .675 \text{ (tension)}$$

$$E_{78} = 3 \times .675 = 2.025 \text{ (tension)}$$

$$C_{78} = .675 \text{ (compression)}$$

$$A_{78} = 2.025 \text{ (compression)}$$

and so on for the succeeding columns, the results coming as tabulated below.

Horizontal or Shear Forces in Columns.—The total shearing force on the top columns is 4 kips, so that, adopting our assumption that this force is uniformly distributed, we shall have the shearing force in each column, acting through the point of contraflexure, equal to $\frac{4}{4} = 1$.

Similarly, in the next columns, the total shearing force = 10.

$$\therefore \text{Shear in each column} = \frac{10}{4} = 2.5 \text{ and so on.}$$

The values for the other columns are shown in *Fig. 1*, and are tabulated below.

Bending Moments in Columns.—To consider the bending moments in the columns, which by symmetry will be the same for *A*, *B*, *C* and *D* at corresponding levels, we consider the deflected form, shown exaggerated in *Fig. 2*, and imagine them cut through at the points of contraflexure at which act equal and opposite forces of magnitude equal to the shearing forces. The top half of the column 8, 9, acts as a cantilever subjected to an isolated load of one kip at its free end, so that the B.M. diagram will be a triangle as shown, the B.M. at the top being equal to $1 \times 6 = 6$ ft. kips.

For the lower half of the top column the diagram will be similar but reversed in sign as shown. The maximum B.M. in the top half of the next column will be $2.5 \times 6 = 15$ ft. kips, thus giving the diagram indicated.

The diagrams can be found in exactly the same way for the other columns, giving the results in the following table.

Column Number.	Vertical Forces (Kips)		Horizontal or Shear Forces (Kips)	Maximum Bending Moment (Ft.-Kips)
	Inside Column.	Outside Column.		
8, 9	0.15	0.45	1.0	6
7, 8	0.675	2.025	2.5	15
6, 7	1.65	4.95	4.0	24
5, 6	3.07	9.225	5.5	33
4, 5	4.95	14.85	7.0	42
3, 4	7.27	21.82	8.5	51
2, 3	10.05	30.15	10.0	60
1, 2	14.55	43.65	12.0	120

Bending Moments on Beams.—Consider any one of the beams, say 6, 6. This beam with the forces acting on it is shown in *Fig. 3*. If it is turned round to a vertical position it will be seen that the beam resembles a column with a number of brackets on each side, downward forces being applied to the brackets on one side and upward forces to those on the other.

The bending moment diagram will be of the form shown: B_{CL} is the B.M. at the left of the point C , while B_{CR} is that at the right of the point.

The bending moment at the point A_6

$$= B_A = 4 \times 6 + 5.5 \times 6 = 57 \text{ ft. kips.}$$

Next consider the point C .

Taking moments just to the left of it, we have:

$$\begin{aligned} B_{CL} &= 4 \times 6 + 5.5 \times 6 + 4.95 \times 16 - \\ &\quad 9.225 \times 16 \\ &= -11.4 \text{ ft. kips.} \end{aligned}$$

Just beyond the point C , we have:

$$\begin{aligned} B_{CR} &= 4 \times 6 + 5.5 \times 6 + 4 \times 6 + 5.5 \times 6 \\ &\quad + 4.95 \times 16 - 9.225 \times 16 \\ &= +45.6 \text{ ft. kips.} \end{aligned}$$

By symmetry we shall have

$$B_{DL} = -45.6, \quad B_{DR} = +11.4; \quad B_E =$$

-57 ft. kips, but we could continue calculating as above. For instance, for B_{DL} we take moments about D .

$$\begin{aligned} B_{DL} &= 4 \times 6 + 5.5 \times 6 + 4 \times 6 + 5.5 \times 6 + 4.95 \times 32 - 9.225 \times 32 - 3.075 \times 16 \\ &= -45.6 \text{ ft. kips.} \end{aligned}$$

The results for other floors are found in exactly the same way, and may be tabulated as shown below.

Points of Contraflexure of Beams.—In the spans AC and ED the points of

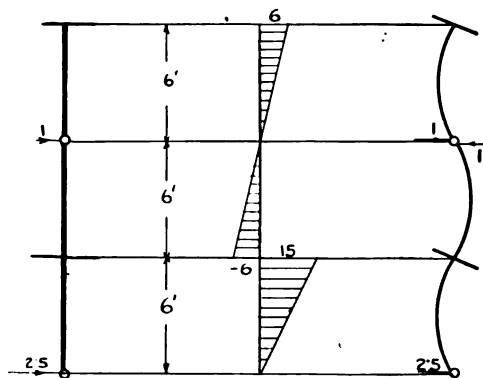


FIG. 2.

contraflexure—i.e., of zero bending moment—will divide the span in the same ratio as the end bending moments.

$$\text{i.e., } \frac{FC}{FA} = \frac{11.4}{57} = \frac{1}{5}$$

$$\therefore \frac{FC}{AC} = \frac{FC}{FA + FC} = \frac{1}{5 + 1} = \frac{1}{6}$$

$$\therefore FC = \frac{16}{6} = 2.67 \text{ ft.}$$

$$\therefore JD = 2.67 \text{ ft.}$$

In the span CD the point H will be at the centre because $M_{CR} = -M_{DL}$.

From the table of bending moments we see that the bending moments in the other spans are in the same proportion, so that the points of contraflexure of all the outside spans is at 2.67 ft. from the inside ends C and D , while those of all the centre spans will be at their mid points.

In connection with these beam and column bending moments, we should keep in mind that for static equilibrium at any intersection of beams and columns the difference in the bending moments of the columns above and below the point must be equal and opposite to the difference in the bending moments in the beams to the right and left of the point.

At the point C_6 , for instance, we have:

$$\begin{aligned} \text{Difference in beam bending moments} &= 45.6 - (-11.4) \\ &= 57 \end{aligned}$$

$$\begin{aligned} \text{Difference in column bending moments} &= -24 - 33 \\ &= -57 \text{ ft. kips.} \end{aligned}$$

Direct Forces or Thrusts in Beams.—Take the top or roof beam 9-9. The thrust in the beam $ED_9 = 4$ kips—1 kip (the shearing force in the column E_9) = 3 kips. The thrust in the beam $DC_9 = \text{Thrust in } ED_9 - \text{the shear in column } D_9 = 3 - 1 = 2$ kips. Similarly the thrust in $CA_9 = 1$ kip.

Now take the next floor 8-8.

The thrust in ED_8 will be $6 + 1 - 2.5 = 4.5$ kips. (This will be followed from Fig. 2, in which we show that the point of contraflexure may be considered as a hinge and equal and opposite forces, to the right and left respectively, considered as acting upon the upper and lower portions.)

Similarly the thrust on DC_8 will be $4.5 + 1 - 2.5 = 3$ kips, and the thrust in $CA_8 = 1.5$ kips.

The values for other floors are shown in the following table:

558

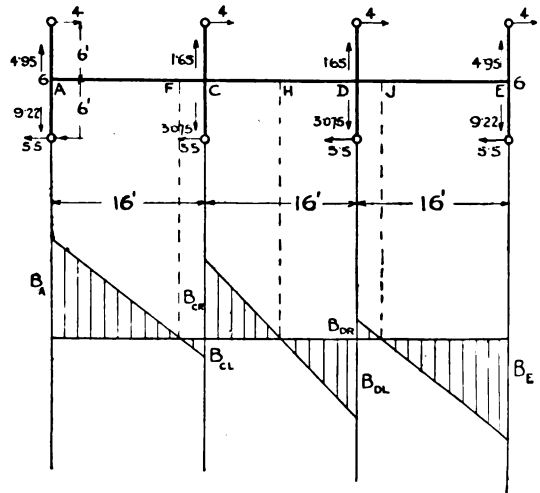


FIG. 3.

Flow Level.	Beam AC			Beam CD			Beam DE		
	Thrust (kips)	B.M. (ft.-kips)		Thrust (kips)	B.M. (ft.-kips)		Thrust (kips)	B.M. (ft.-kips)	
		Left	Right		Left	Right		Left	Right
9	3	6	- 1.2	2	4.8	- 4.8	1	1.2	- 6
8	4.5	21	- 1.25	3	16.8	- 16.8	1.5	4.25	- 21
7	4.5	30	- 7.8	3	31.2	- 31.2	1.5	7.8	- 39
6	4.5	57	- 11.2	3	45.6	- 45.6	1.5	11.2	- 57
5	4.5	75	- 15	3	60	- 60	1.5	15	- 75
4	4.5	93	- 18.6	3	74.4	- 74.4	1.5	18.6	- 93
3	4.5	111	- 22.2	3	88.8	- 88.8	1.5	22.2	- 111
2	6	180	- 36	4	144	- 144	2	- 36	- 180

In using this method we should keep in mind the fact that it does not profess to be exact. The relative stiffness of the beams and columns will have some effect upon the result, but it is practically impossible to make adequate allowance for this, and the above method may be taken as the best simple one to adopt.

MEMORANDA.

Reinforced Concrete Regulations.—At the first meeting of the London County Council after the summer recess the Building Acts Committee reported that the Local Government Board had allowed the regulations made by the Council on July 6th, 1915, under Section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete and with respect to the use and composition of reinforced concrete in such construction. The Board has fixed January 1st, 1916, as the date upon which the regulations shall come into operation.

The Microscope in the Study of Concrete.—We regret that owing to lack of space we are compelled to hold over the concluding article of this series until next month.



CONCRETE ROADS.

The application of concrete for roads is arousing considerable interest, especially among borough engineers and surveyors, and we have from time to time published particulars of concrete road work both in this country and in the United States. We would draw the special attention of our readers to the subjoined notes on roads in different parts of England.—ED.

ROBERTS LANE, SALTNEY, CHESTER.

IN our August number of last year we published a description, with photographs, of one of the first reinforced concrete roads that had been constructed in this country. We allude to the Roberts Lane road at Saltney, in the environs of Chester, made in 1912, under the personal supervision of Mr. Matthews Jones, the Borough Surveyor. Briefly described, this road, which is 950 ft. in length and 20 ft. broad, was excavated to a depth of 8 to 10 in., then on top of the clay subsoil 2 in. of cinders was placed to bind it. On this $2\frac{1}{2}$ to $3\frac{1}{2}$ in. of concrete, then across the road the reinforcement of the Expanded Metal Co.'s rib expanded steel No. 8 was laid. Covering this was concrete to a depth of $3\frac{1}{2}$ to $4\frac{1}{2}$ in. The concrete was made of five parts of broken granite $\frac{3}{4}$ in. to small 2-in. (mixed) and sharp sand, to one of cement, "J. B. White Bros." brand. The road was closed for about three weeks, and for part of the time kept well watered. After being opened for traffic, the road remained for nearly a year without anything being done to it, when it was tar sprayed and sprinkled with granite chippings, at a cost of $1\frac{1}{2}$ d. per yard. This treatment is carried out once a year, so the cost of maintenance is a very light one.

When this road was inspected in May of 1914 the surface was in perfect condition. In the spring of this year the tar spraying, etc., was done, and when seen again in August last the surface was still in complete order, and there was not the slightest sign of the dressing lifting in any way. On this point it is well worth remembering that when the tar spraying and chippings were first put down, the concrete was thoroughly dry and set.

The traffic on this road fifteen months since was computed to be about 60 tons per day—not heavy—but undoubtedly this has increased, and will still further do so as the property in and around this district is developed. The original cost of this road, including the excavation, was 3s. 10d. per yard, and the cost of maintaining the surface has already been mentioned.

THE NEW ROAD AT WHITEFRIARS, CHESTER.

This road was laid in August, 1914, by Mr. Matthews Jones, but not quite under the same conditions as those at Roberts Lane. Its length is 510 ft., width 15 ft., and there is quite a fair amount of heavy traffic passing over it. Originally this road was made with wooden blocks on a concrete foundation, which from time to time had been broken up for repairs to gas and water mains, so that the concrete foundation was not in good condition. Two inches of concrete was put in, and then, across the road, the metal reinforcement—which in this instance was the B.R.C. Fabric No. 9, supplied by the British Reinforced Concrete Engineering Co.—was placed, and on this a further 4 in. of concrete. This latter was again 6 to 1, composed of four parts of Welsh granite, graded from $\frac{1}{4}$ to 1 in., two parts of fine sharp sand, and one part of cement, "J. B. White Bros." Brand. On completion, the road was closed for three weeks, kept well watered, and then opened for traffic. Later it was tar sprayed and granite chippings put on.



Fig. 1. Road in Course of Construction. Whitefriars Road, Chester.
CONCRETE ROADS.

The water and gas mains are under this road, but before the work was started notice was given to the companies to see that their pipes were in perfect order. The cost of this road, including the excavation, was 4s. 10d. per yard, or 1s. per yard more than that at Roberts Lane. It must, however, be borne in mind that the cost of all materials and labour had considerably increased. Still, it is a cheap road, and the surface, when seen at a recent inspection, presented no indications of wear.



Fig. 2. View of Completed Road. Whitefriars Road, Chester.
CONCRETE ROADS.

As regards reinforced concrete roads in Chester, there is little doubt that had it not been for the war and the economies it has entailed several others would have been constructed.

ROAD AT MOUNTNESSING, ESSEX.

This road is one of peculiar interest, inasmuch as the sides of it are composed of reinforced concrete, whilst the centre track, a little over 11 ft., is of water-bound macadam. Mountnessing is on the main road between London and Colchester. The length of the section under notice is about three-quarters of a mile, and the width 22 ft. 4 in. An excavation was made at the sides to a depth of 13 in., but other than this it was about 7 in.

The subsoil in this locality is clay, but as in early days this road from time to time had been dressed with gravel it was the latter on which the work of the new road was started. First of all a layer of 3 in. of ashes was placed on the loose gravel, then $1\frac{1}{2}$ in. of concrete, on this the metal reinforcement, No. 9 B.R.C. Fabric supplied by the British Reinforced Concrete Engineering Company, and on the top of this $3\frac{1}{2}$ in. of concrete. On either side of the road is a concrete kerb reinforced with one steel rod sunk to a depth of 10 in. This stands 3 in. above the surface of the road, and is 5 in. wide. The kerbing was made and placed as the road progressed. The width of the reinforced concrete on either side of the macadam is 5 ft. 7 in., and in support of the macadam is a concrete abutment running down from the level of the road to a depth of 8 in. As regards the water-bound macadam, which extends to a depth of 3 in., it is the old material scarified and a thickness of 2 in. of new macadam rolled and water-bound.

The whole of the concrete was made up of 3 parts of crushed ballast graded so as not to exceed 1 in., 1 part of washed sand, and 1 part of cement, "Hilton Anderson" brand. When the road is completed and opened for traffic it will, after a time, when the concrete is thoroughly dry, be lightly tar sprayed and have $\frac{3}{8}$ -in. crushed flint chippings put on its surface.

It should be observed that the primary reason for adopting reinforced concrete in this case was to provide a foundation to this portion of the road, on

which the sides were very bad. The question of leaving the concrete to act as a wearing surface was in the nature of an afterthought.

Fig. 3 is a diagram of the road, showing in a section from kerb to kerb the position of the concrete reinforcement, macadam, etc.

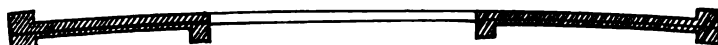


Fig. 3. Section : Road at Mountnessing, Essex.
CONCRETE ROADS.



Fig. 4. Road under Construction, Mountnessing, Essex.
CONCRETE ROADS.

This work was planned and arranged with the approval of the Road Board by Mr. Percy J. Sheldon, county surveyor, and Mr. Alfred Lyddon, deputy county surveyor, under whose personal supervision it was carried out, the contractors being Messrs. W. and C. French, Buckhurst Hill.

MARINE DRIVE, EXMOUTH.

What will in all probability be a fine example of reinforced concrete road construction is now in progress in connection with the Marine Drive, Exmouth.

The length is one mile, and of a total width of 65 ft.—viz., carriageway 39 ft. 6 in., footpaths 5 ft. 6 in. and 20 ft. wide respectively.

The carriageway will be concrete 5 in. thick on a sand underbed between concrete kerbs. Felt expansion joints will be placed every 24 ft.

The reinforcement is the Expanded Metal Co.'s diamond mesh expanded steel placed $2\frac{1}{2}$ in. from the surface.

The concrete is composed of six parts of beach gravel, fine and coarse proportionately, to one part Portland cement of British Standard Specification.

A trial portion was laid in May last, and tar sprayed one coat. Traffic was put on three weeks later and restricted to a width of 8 ft. of the carriageway in order to seriously test the concrete. Considerably more than 1,000 tons has



Fig. 5. Section: Marine Drive, Exmouth.
CONCRETE ROADS.

passed over this narrow portion, and there is not the slightest sign of wear.

Fig. 5 shows a cross section of the road : and the photograph Fig. 6 shows the trial portion referred to with concrete kerbs made on the spot.

Half a mile of this Marine Drive is an excellent example

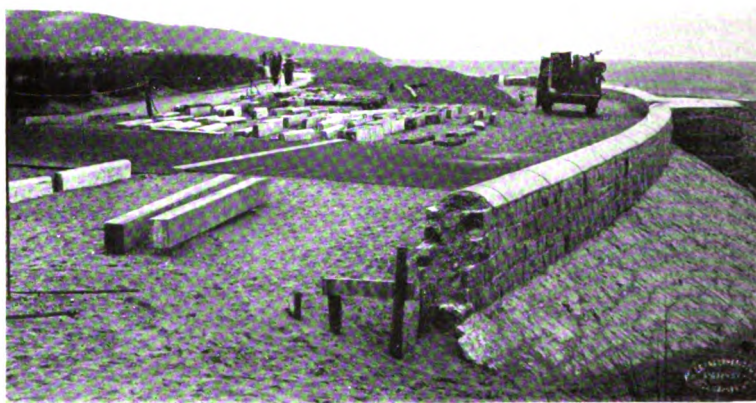


Fig. 6. Marine Drive, Exmouth.
CONCRETE ROADS.

of the application of reinforced concrete to engineering ends. The piling is concrete reinforced with plain bars, the slope and parapet also of concrete reinforced with expanded metal and faced with limestone, and, as stated above, the roadway is of reinforced concrete.

A further notice of this work when completed will doubtless be of considerable interest.

The engineer for the whole of the road construction and the half mile of reinforced concrete sea defence is Mr. Samuel Hutton, engineer and surveyor to the Exmouth Council, the engineers for the remaining half mile of defence works (bulk concrete wall) are Messrs. Douglas, Lewis and Douglas, 15, Victoria Street, S.W., and the resident engineer for the whole of the work is Mr. J. T. Pierce.

The contractors are Messrs. Tom Wilkinson and Co., Poole, and Messrs. Wilson and Co., Exmouth.

PADSTOW STATION.—REINFORCED CONCRETE ROAD.

This road is on the Fish Quay at Padstow, London and South-Western Railway, and was laid about twelve months since. Its length is 107 ft. and width 20 ft.



Fig. 7. Padstow Station.—Fish Quay Road under Construction.

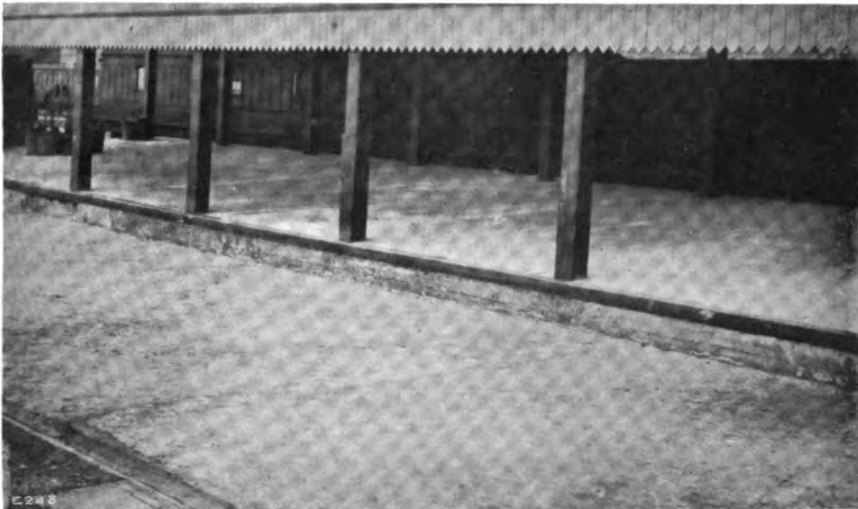


Fig. 8. View of Finished Road. Fish Quay Road, Padstow.
CONCRETE ROADS.

At times it is subjected to very heavy traffic, but during the period which has elapsed since it was thrown open it has so well stood the test of practical use that there is no sign of wear. In the formation of this road there was first laid 2 in. of cinders, on this 3 in. of concrete, consisting of

five parts of broken granite $\frac{3}{4}$ in. to small 2 in. (mixed) and sharp sand to one of cement, "J. B. White" brand. Then placed across the roadway on the top of the concrete was a reinforcement of the Expanded Metal Company's rib steel No. 6. Covering this was concrete of the same character as previously described to a depth of 3 in., the surface being roughly smoothed over. For three weeks the road was closed to traffic, and for the first nine days the concrete was kept well watered.

The illustrations *Figs. 7 and 8* show the road in course of construction and when finished.

MEMORANDA.

First Concrete Motor Track in America.—The motor racing tracks hitherto laid down in America have in most cases had wood surfaces, but the Twin City speedway, which has recently been completed, is the first concrete-surfaced race-course to be laid down in the States. The track measures two miles, and is oval in shape. The width is 80 ft. for 1,577 ft. in front of the judge's stand, and on the turns and back stretch is 60 ft. The straight is 285 ft. long, practically flat, with an angle of only 1 ft. There are easement curves of 800 ft. from the ends of the two stretches entering into the high circle or leaving it, at 8 ft. high. These four easements allow the drivers to rise gradually into the total banking of 14.98 ft. at the outside from the ground at each end of the oval, and to drop down again into the straights. The change from the end of the easements into the high curve is a one per cent. grade. There are 40 pits, each 8 ft. by 15 ft., adjacent to the 30 in. track-retaining wall, separated by 2 in. by 4 in. wood partitions with wood backs, equipped with a bench seat at the rear and sand floor. Seating capacity is provided for 71,616. There are four grand-stands in front of the pits in the finishing straight, a fifth is situated at the end of the back straight, and another at the end of the top of the second curve. Motor-cars enter the grounds at six points, and to the oval by three subways. Behind the main grand-stand is free accommodation for 15,000 cars, and inside the oval there is space reserved for 22,000 cars. The land occupied by the new track and enclosures is $342\frac{1}{2}$ acres, and altogether the Twin City course is the most ambitious of the many American speedway propositions. Its future will be followed with interest.—*Motor*.



Concluded from previous issue. The first three parts appeared in August, September and October.—ED.

PART IV.

PILLARS AND OTHER STRUTS.

Strut Defined.—94. The term "strut" when used in these regulations shall be deemed to include any compression member at any angle.

Pillar Defined.—95. The term "pillar" when used in these regulations shall be deemed to include any pillar, pier, post, column, detached support or any other vertical compression member.

96. Pillars shall be designed on the assumption that the concrete and the vertical bars are shortened in length in the same proportion.

Ratio.—97. In calculating the strength of a pillar, the maximum ratio of length to gyration radius or the maximum ratio of length to effective diameter shall be taken.

Length.—98. The length shall be measured between the lateral supports, irrespective of any splayed work in excess of that allowed as part of the beam by regulation 70.

Effective Diameter.—99. The effective diameter shall be measured to the outside of the outermost vertical reinforcement, and shall be measured in the direction of the lateral supports which determine the length of the pillar.

Reinforcement.—100. All pillars shall be provided with vertical and lateral reinforcement.

Binding.—101. Each pillar with rectilinear laterals (binding) shall have at least four lines of vertical reinforcement throughout its entire length.

102. Each pillar with curvilinear laterals (binding) shall have at least six lines of vertical reinforcement throughout its entire length.

103. The least diameter of rectilinear laterals (binding) shall not be less than $\frac{1}{8}$ of an inch.

104. The least diameter of curvilinear laterals (binding) shall not be less than $\frac{1}{8}$ of an inch.

105. (a) The pitch of the laterals (binding) shall not exceed $\frac{1}{6}$ of the effective diameter of a pillar at any part of its length or 16 times the diameter of the least vertical bar.

(b) At the ends of a pillar for a length equal to one and a half times the effective diameter the pitch of the laterals (binding) shall not exceed $\frac{1}{6}$ of such effective diameter.

106. The lateral reinforcement (binding) shall be firmly secured at each end.

107. The volume of lateral reinforcement (binding) shall not be less than 0.5 per cent. of the volume of the hooped core.

Vertical Bars.—108. The diameter of vertical bars shall not be less than $\frac{1}{2}$ inch or greater than 2 inches.

109. The total cross-sectional area of the vertical reinforcement in any pillar shall not be less than 1.0 per cent. of the area of the hooped core.

Joints.—110. Joints in the vertical reinforcement of pillars shall only be made at or adjacent to a floor level or other point of lateral support.

111. In all joints in the vertical reinforcement there shall be provided an overlap at least equal to twenty-four times the diameter of the upper bar.

In cases where there may be tension in the pillar the ends of the bars shall be formed as required by regulation 48.

Rectangular Pillars.—112. In the case of rectangular pillars in which the ratio between the greater and the lesser diameter exceeds one and a half, the cross section of the pillar shall be subdivided by cross-ties, and the number of vertical bars shall be such that the distance between the bars along the longer side of the rectangle shall not exceed the distance between the bars along the shorter side of the rectangle.

Notation for pillars, struts and other compression members.

Notation.—113.

A = effective area of the pillar, i.e., the area bounded by the lateral reinforcement (binding) measured to the inside of the hooping.

A_b = cross-sectional area of one bar of the binding or lateral reinforcement.

A_v = area of the vertical reinforcement.

c = permissible direct compressive stress. (See regulation 42.)

d = effective diameter. (See regulation 99.)

f = form factor, depending upon form or type of binding or laterals. (See table in regulation 118.)

g = gyration radius of the area $[A + (m-1)A_v]$ about an axis passing through the centroid of the area.

i = increased stress permissible in the core of a pillar suitably hooped.

l = actual length of the pillar as defined in the regulation numbered 98.

m = modular ratio = $\frac{E_s}{E_c}$. (See regulation 54.)

P = permissible load or pressure on pillars with both ends fixed and with a ratio of virtual length to gyration radius not exceeding 45. (See first column of figures in regulation 122.)

p = percentage of volume of binding or lateral reinforcement with respect to the volume of hooped core in any given length of pillar = 100 V_r .

p_b = pitch of the binding or lateral reinforcement.

s = spacing factor depending upon the spacing or pitch of the binding or laterals. (See table in regulation 118.)

V_r = volume ratio or ratio of the volume of binding or lateral reinforcement to the volume of the hooped core in any given length of pillar. (See regulations 116 and 117.)

v = virtual length of the pillar, strut or other compression member for different conditions of the ends. (See regulations 122 and 124.)

Fixed Ends.—114. A pillar, or other strut, shall be deemed to have fixed ends when the ends are sufficiently secured to other parts of the construction having such rigidity as will maintain the axis at the ends in its original position and direction under all loads less than the crippling load.

Formulae.—115. In a pillar with fixed ends the stress in the concrete in the area bounded by the lateral reinforcement shall not exceed

c , in a pillar with the minimum of hooping or binding,

i , in a pillar with more than the minimum of hooping or binding, where

$$i = c [1 + fs V_r]$$

116. The required ratio of the volume of binding or lateral reinforcement to the volume of the hooped core may be obtained from the equation—

$$V_r = \frac{i - c}{cfs}$$

117. The value of V_r actually provided, in either round or square pillars, may be ascertained by direct measurement or from the equation—

$$V_r = \frac{4 A_b}{d p_b}$$

118.

Table showing the value of f_s .

Form of lateral reinforcement or binding.	Form factor f .	Spacing of laterals throughout the length of the pillar (in terms of diameter of hooped core).	Spacing factor s .	Value of f_s .
Helical (curvilinear on plan) ..	1.0	0.2d or less	32	32
Helical " " ..	1.0	0.3d	24	24
Helical " " ..	1.0	0.4d	16	16
Circular hoops	0.75	0.2d or less	32	24
Circular hoops	0.75	0.3d	24	18
Circular hoops	0.75	0.4d	16	12
Rectilinear	0.5	0.2d or less	32	16
Rectilinear	0.5	0.3d	24	12
Rectilinear	0.5	0.4d	16	8
Rectilinear	0.5	0.5d	8	4
Rectilinear	0.5	0.6d	0	0

119. Notwithstanding any other provision in these regulations the increased stress in the concrete of pillars shall not exceed one-third of the ultimate compressive resistance at four months as given in regulation 159.

120. The working stress in the vertical reinforcement shall not exceed m times the stress in the concrete. (See regulation 54.)

121. The permissible load or pressure on pillars with both ends fixed and with ratios not exceeding those in the first column of figures in regulation 122 shall be obtained from either of the equations—

For pillars with the minimum of hooping or binding—

$$P = c [A + (m - 1) A_r]$$

For pillars with more than the minimum of hooping or binding—

$$P = i [A + (m - 1) A_r]$$

122. For pillars the permissible load or pressure shall be obtained from the following table—

For the maximum ratio of <i>virtual</i> length to gyration radius	$\frac{v}{g} =$	45	54	63	72	81	90
For the maximum ratio of <i>virtual</i> length to effective <i>diameter</i> for square pillars	$\frac{v}{d} =$	15	18	21	24	27	30
For the maximum ratio of <i>virtual</i> length to effective <i>diameter</i> for round pillars	$\frac{v}{d} =$	12	15	18	21	24	27
Permissible load	P	0.8 P	0.6 P	0.4 P	0.2 P	Nil.	

For other ratios higher than those in the first column of figures the stress shall be proportional to the above.

Struts.—123. Struts shall be subject to all the rules applying to pillars, and the combined stresses at any part shall not exceed the permissible stresses for pillars of like ratios. (See regulations 44, 45, 46.)

As to Conditions of Ends of Struts and Pillars.—124. For the different conditions of the ends of pillars and other struts mentioned in the following table, the

permissible load shall be ascertained by using the corresponding value of the virtual length in such table in conjunction with regulations 121 and 122—

Condition of axis at ends.			Virtual length, v .
Both ends fixed in position and direction	..	..	$v = l$
One end fixed in position and direction and One end fixed in position but not in direction	..	..	$v = 1.4 l$
Both ends fixed in position but not in direction	..	..	$v = 2l$
One end fixed in position and direction and One end not fixed in position and direction	..	..	$v = 4l$

Arches. (Also see Regulations 44, 45, 46.)—125. In the case of arches or other similar constructions the combined stresses at any part shall not exceed the permissible stresses set out in regulations 42 and 43.

PART V.

WALLS.

Thickness of Walls.—126. Where the dead loads and superimposed loads of, in or upon a building are transmitted to the foundations by a series of reinforced concrete pillars, beams, arches or other constructions designed and constructed in accordance with these regulations, any external enclosing walls of reinforced concrete between such pillars may be of any thickness not less than 4 in., provided that such enclosing walls are designed and constructed in accordance with these regulations to resist any loads and pressures they may have to carry.

127. In any case where any wall or part of a wall is intended to support vertical loads or resist lateral pressures, it shall be of such thickness as may be necessary to keep the stresses within the limits prescribed by these regulations for the construction of pillars, beams and other members.

128. When portions of the external walls between the reinforced concrete pillars and beams are constructed of brickwork, stonework, or plain concrete, such portions of walls shall be of a thickness not less than $8\frac{1}{2}$ in. for the topmost 20 ft. of their height and not less than 13 in. for the remainder of their height below such topmost 20 ft.

When such portions are constructed of hollow blocks, the blocks shall conform to the following requirements—

(a) The aggregate thickness, including any cavity, shall be at least $8\frac{1}{2}$ in., measured at right angles to the face of the wall.

(b) The sides shall be at least 2 in. thick.

(c) The ends and any interior partitions shall be at least 1 in. thick.

(d) The clear unsupported span of any part of any side, end, or interior partition shall not be more than four times the least thickness of such part.

(e) The aggregate thickness of the material shall be at least 6 in., measured at right angles to the face of the wall.

Provided that a less thickness shall be allowed in any case in which under the London Building Act, 1894, such less thickness is prescribed, and provided that in any case in which an external wall or portion of an external wall is not supported or carried or secured by the reinforced concrete skeleton framed construction within the limit of height and length prescribed by the First Schedule to the London Building Act, 1894, for the purpose of determining the thickness of walls, such external wall or portion of external wall shall be of a thickness not less than that prescribed by such schedule.

Facing.—129. All walls and facing materials shall be incombustible and shall be securely connected to the pillars, beams, floors, and other contiguous parts of the reinforced concrete construction.

Pressure on Panels.—130. Each panel in any external wall shall be designed to resist safely a horizontal pressure equivalent to a static pressure of at least 30 lb. per sq. ft. assumed to be acting uniformly over the area of one panel from either side. (See regulations 21 and 22.)

Openings.—131. The aggregate area of openings in external walls constructed in accordance with these regulations shall not exceed in a wall of any storey above the ground storey two-thirds of the whole area of such wall, and the aggregate width of such openings in such a wall shall not exceed three-quarters of the whole length of such wall.

Party and Division Walls.—132. Party walls and division walls constructed in reinforced concrete in accordance with these regulations shall be of such thickness as may be necessary to comply therewith, but in no part shall such walls be of less thickness than 8 in.

133. Provided that any such party wall between a building constructed in reinforced concrete and a building of the warehouse class constructed in accordance with the Rules of the London Building Act, 1894, or as a steel-framed building, shall not be in any part less than 13 in. in thickness.

Mortar.—134. All brickwork, stonework, and plain concrete shall be executed in Portland cement mortar. The mortar shall be in accordance with the Council's by-laws from time to time in operation.

Pressure on Brickwork.—135. The pressure on any brickwork supporting reinforced concrete work shall not exceed the following :—

Brickwork.	Tons per sq. ft.
Blue brick in cement mortar.	12
Hard brick (including London stock) in cement mortar	8
Ordinary brick in cement mortar	5

Ratio of Height to Thickness.—Such brickwork shall not have a height without proper lateral supports of more than six times its least thickness, but any such brickwork with proper supports may have a height between such lateral supports not more than twelve times the least thickness of such brickwork. Such thickness shall in no case be less than $13\frac{1}{2}$ in.

PART VI.

FOUNDATIONS.

Pressure on Foundations.—136. The pressure of foundations on the natural ground shall not exceed the following :—

Nature of ground.	Tons per sq. ft.
Natural bed of soft clay or wet or loose sand	1
Natural bed of ordinary clay or confined sand	2
Natural bed of compact gravel, London blue clay or chalk	4

137. The pressure on plain concrete in foundations shall not exceed twelve tons per square foot. The plain concrete shall be in cement and its quality shall be at least equal to that required by the Council's by-laws from time to time in operation.

PART VII.

PROTECTION.

138. The cover shall be measured from the outer surface of the concrete to the outer surface of the metal reinforcement.

End Cover.—139. The end cover to be provided beyond the anchored end of a bar coming within the provisions of regulation 48 (c) shall not be less than 2 in. and not less than twice the normal diameter of the bar to be covered.

Pillars.—140. The cover of any vertical bar in a pillar shall not be less than $1\frac{1}{2}$ in. and not less than the diameter of such vertical bar.

Beams.—141. The cover of any longitudinal bar in a beam shall not be less than 1 in. and not less than the diameter of such longitudinal bar.

Slabs.—142. The cover of tensile, compressive, shear or any other reinforcement in

slabs shall not be less than one half inch, and not less than the diameter of the bar to be covered.

Other Members.—143. The cover of any reinforcement in other members shall not be less than 1 in. and not less than the diameter of the bar to be covered.

PART VIII.

MATERIALS AND TESTING.

Cement.

Cement Standard.—144. All cement used shall be Portland cement of slow setting quality and shall be in accordance with the British Standard Specification from time to time in operation.

Weight.—145. The quantity of cement shall be determined by weight, and ninety pounds shall be deemed to be the equivalent of one cubic foot.

Sand.

Sand.—146. The sand shall be clean and gritty. It shall be composed of hard silicious grains, or of materials permitted under regulation 150. It shall be free from clay or any animal, vegetable, or bituminous matter.

147. All sand shall pass through a mesh three-sixteenths of an inch square measured in the clear.

148. The sand shall be separated from the coarse material before the materials are measured.

Coarse Material.

Coarse Material Defined.—149. The term "coarse material" means all the ingredients of the concrete except the sand, the cement, and the water.

Permissible Materials.—150. The coarse material shall consist of clean Thames or pit-ballast or gravel, hard stone, such as granite, basalt, trap rock, or other hard and equally suitable material. It shall be free from clay or any animal, vegetable, or bituminous matter.

Prohibited Materials.—151. The following materials shall not be used with the sand or coarse material in the composition of the concrete under these regulations:—

(a) Coal residues, including clinkers, cinders, ashes, coke breeze, pan breeze, slag, and other similar material.

(b) Blast furnace slag, copper slag, forge breeze, dross, and other similar material.

(c) Sulphates, including plaster of Paris, and other similar material.

(d) Limestones, magnesian limestones, marbles, and other calcium carbonates.

Washing.—152. Unless quite clean, all coarse material shall be thoroughly washed.

Sizes.—153. The coarse material shall be of such a size as will pass through a mesh three-quarters of an inch square measured in the clear and be retained on a mesh three-sixteenths of an inch square measured in the clear.

154. The coarse material shall be varied in size as much as possible between the limits of size allowed for the work, but subject to the provisions of regulation 142 it shall not be larger than such as can pass between the bars forming the reinforcement or between the reinforcement and the centering.

Wetting.—155. The coarse material if of a porous nature shall be thoroughly wetted before being mixed with the other materials.

Volumes.—156. The volume of mortar shall be in excess of what would be required to fill completely the interstices and voids of the coarse material.

157. The volume of the sand shall not be more than twice the volume of the cement.

158. The volume of coarse material shall not be more than twice the volume of sand.

Concrete.

Proportions and Ultimate Resistance.—159. The concrete shall be composed of cement, sand, and coarse material in one of the following or intermediate proportions, and the ultimate compressive resistance shall not be less than that specified for the proportion adopted—

Proportion by volume.			V	Ultimate compressive resistance in lbs. per sq. in.		Value of one-fourth of u_4
Cement.	Sand.	Coarse material.		1 month after mixing = u_1 ,	or, 4 months after mixing = u_4 ,	
1	2	4	6	1,600	2,400	600
1·2	2	4	5	1,800	2,600	650
1·5	2	4	4	2,000	2,800	700
2	2	4	3	2,200	3,000	750

(Also see regulations 42, 145, 148, 156, 157, 158, 162.)

160. For determining the resistance of concrete, tests shall be made on cubes of not less than 4 in. each way, or cylinders of not less than 6 in. each way.

Tests.—161. The conditions accompanying the preparation, setting, maturing, and actual testing of the cube or other test piece shall, as far as possible, conform to the conditions that would obtain in the actual execution of the reinforced work.

Ultimate Resistance.—162. The ultimate compressive resistance of concrete of materials mixed in intermediate proportions may be estimated from the following equation—

$$u_1 = 2,800 - 200 V, \text{ or} \\ u_4 = 3,600 - 200 V, \text{ where :—}$$

u_1 = ultimate compressive resistance at one month, in pounds per sq. in.

u_4 = ultimate compressive resistance at four months in pounds per sq. in.

V = Volume of the sand plus the volume of the coarse material, per volume of cement, each measured separately and including the voids proper to each material.

Mixing.—163. All three materials shall be thoroughly mixed dry in batches, and then again thoroughly mixed in batches after wetting, subject to the requirements of regulation 155.

Salt water shall not be used for the concrete.

Placing.—164. The concrete shall be placed in its final position before initial set has taken place.

In the case of beams, pillars, and walls the thickness of the layers of loose concrete shall not exceed 3 in. before ramming.

Ramming.—As soon as possible after mixing, the concrete shall be properly rammed into the moulds in such a manner and under such conditions as will secure a compact mass, without voids and of the greatest possible density for the proportions used.

Steel.

Steel Standard.—165. All metal reinforcement shall be of steel which shall comply with the British Standard Specification for structural steel for bridges and general building construction from time to time in operation.

Cleaning.—166. All metal for reinforcement shall be cleaned of all scale, dust, and loose rust immediately before depositing the concrete.

No Welding.—167. Butt or scarf welding shall not be employed in any tensile reinforcement.

Tests.—168. The builder or other person directing the work to be executed shall, for the purpose of due supervision of the construction of the building, furnish the District Surveyor with reasonable proof as to the quality of materials to be used in such construction, and shall make any tests which shall be reasonably necessary.

TESTS AND TESTING.

Tests and Testing—169. If at any time during the construction or within two months after the completion of the reinforced concrete construction it is found necessary to test any part of such construction by reason of any sign of weakness or faulty work appearing in the construction, the builder or other person causing or directing the work to be executed shall make such tests, and, if the tests show the work to be faulty, it shall be reconstructed and reinstated in accordance with these regulations.

Deflection.—170. The total deflection of beams or slabs freely supported and uniformly loaded, and subject to the permissible working stresses, shall not exceed

1/600th of the span, when the span is 20 times the effective depth, and shall be in proportion for other ratios of span to depth, and for other conditions of ends and stress and loading.

Test Load.—171. The superimposed test load on any floor, roof, or other structure shall be not more than one and a half times the superimposed load for which such floor, roof, or other structure has been designed. The superimposed test load on any beam, slab, or other similar member which has been exposed to frost during the first week of hardening shall be not less than one and a half times the superimposed load for which such beam, slab, or other member has been designed.

172. Loading tests shall not be made until the expiry of ninety days from the date of laying the concrete.

PART IX.

FORMWORK OR CENTERING.

Formwork Defined. 173. For the purpose of these regulations, the terms "formwork" or "centering" shall include all forms, moulds, sheeting, shuttering, planks, poles, posts, shores, struts and strutting, ties, uprights, walling, and all other temporary supports to the concrete during the process of setting.

Rigidity.—174. The formwork or centering shall be of such dimensions and so constructed as to remain rigid during the laying, ramming, and setting of the concrete.

Strutting.—175. The vertical strutting shall be maintained continuous through the lower storeys to the foundations or to other floors or beams which are sufficiently set to afford the required support without injury to the construction.

Removal.—176. All formwork or centering shall be removed without shock or vibration.

177. Before the formwork or centering under any beam or floor slab is removed the pillars below such beam or floor slab shall be partially stripped so that the pillars may be examined on all sides.

PART X.

WORKMANSHIP.

Placing.—178. All reinforcement shall be placed and maintained in the position shown on the drawings.

Continuity of Work.—179. The concreting in any member shall be carried out as continuously as possible.

Recommencement of Work.—180. Where work has to be recommenced on a surface which has hardened, such surface shall be well hacked, swept clean, thoroughly wetted, and covered with mortar composed of equal volumes of cement and sand.

Dry Weather.—181. Concrete laid during dry weather shall be protected against too rapid drying. During the first week of hardening it shall be kept damp by means of wet sacking or other methods or by watering daily (Sundays and holidays included).

Cold Weather.—182. Concrete shall not be laid when the temperature is below 4 degrees Centigrade (39 degrees Fahr.), and shall be protected when necessary.

Frost.—183. Concrete or mortar which has been frozen shall not be used.

184. The concrete or mortar in any beam, slab, strut, or other similar member which has been exposed to frost during the first week of hardening shall be removed or such members shall be tested as provided for in regulations 169 to 172.

Cutting.—185. No cutting for piping or any other purpose shall be done which would reduce the strength of any part of the structure below the standard required by these regulations.

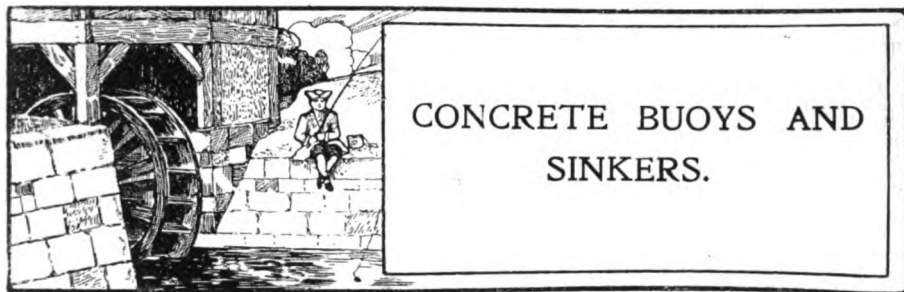
Strips and Blocks.—186. Blocks or strips of hard wood, coke breeze, or other equally fire-resisting material may be embedded in the concrete and used solely for fixing purposes, provided that—

(a) they do not reduce the strength of any part of the structure below the standard required by these regulations;

(b) the area of such blocks or strips at any given cross-section is not included in the calculated compression area of any beam, slab, pillar, or other constructional member.

Inlaid Materials.—187. Soft wood or other equally combustible material shall not be embedded in the concrete.

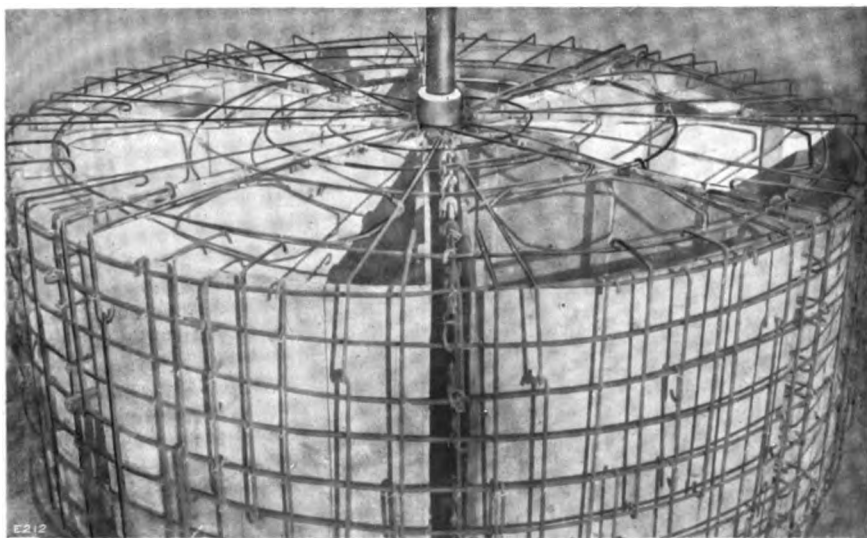
Overlaid Materials.—188. Wood or other combustible material may be placed on or over the surface of the concrete provided that any voids or hollow spaces between the combustible and incombustible materials be filled up with materials of an incombustible nature.



The question of concrete buoys is one of considerable interest, and we publish below a few particulars of a sinker made for use in the Mediterranean.—ED.

FLOATING structures of reinforced concrete in the form of caissons have been made from time to time for certain difficult foundation works and for barges, of which quite a number are now in use for river work, but reinforced concrete has not yet been used to any great extent in the making of buoys.

In July 1913 an article appeared in this JOURNAL describing a concrete buoy made in Jamaica, and which was believed to be the first of the kind. Since then there have been, no doubt, several made in various parts of the world,



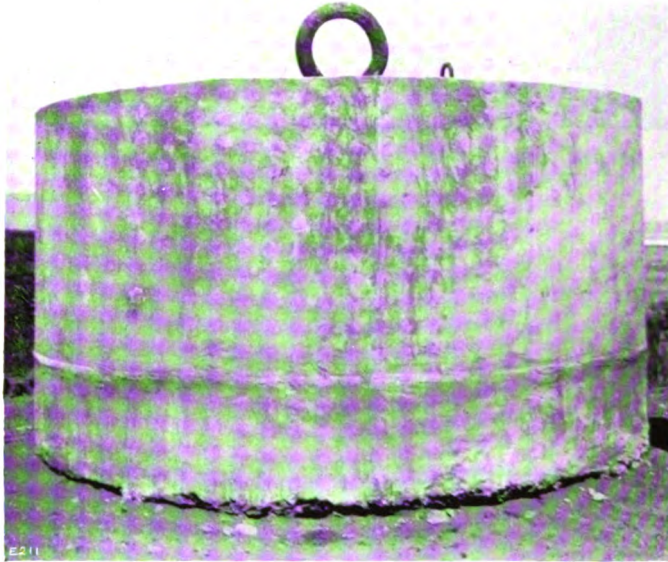
Floor of Sinker laid. Internal casing or mould erected and outer armouring completed ready to receive external casing.

CONCRETE BUOYS AND SINKERS.

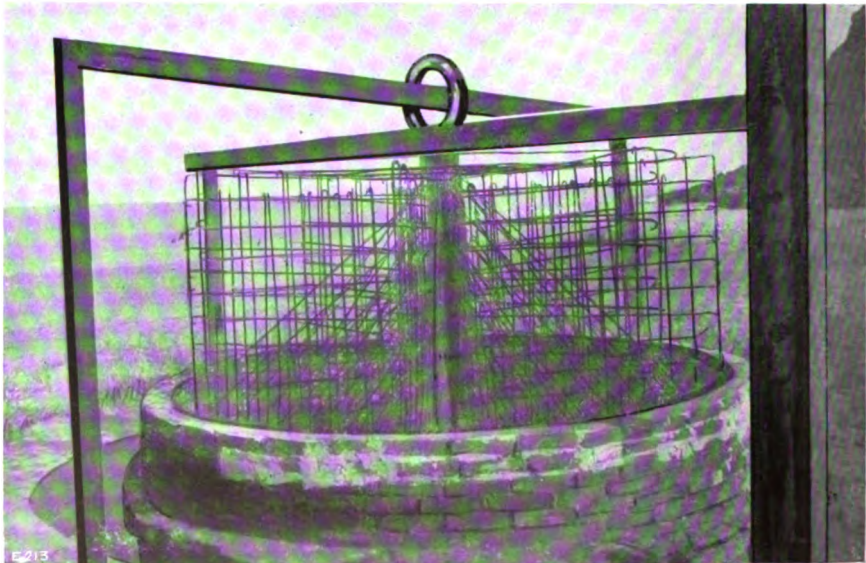
but up to the present they have not been adopted, or at any rate not to any large extent, in England.

Buoys being used for many different purposes, such as for marking channels, shoals, or rocks, or for mooring ships in open waters, docks, and harbours, it follows that great variations occur in design, size, and even in the material used in their construction.

At the present time by far the greater number of buoys are made of steel, though there are certain ports where wooden buoys are still found to be the more practicable; and there is at least one port where 150 concrete buoys are in use with most satisfactory results.



The Finished Sinker.



Bulkhead Armourings and Mooring Eye in place ready for concreting floor. (Dwarf brick wall in plaster found to be most economical as outer mould).

CONCRETE BUOYS AND SINKERS.

It is not suggested that concrete can take the place of steel in all cases, but there are harbours and ports in England where they might be introduced with great advantage.

The outstanding advantage of concrete is that it requires no periodical painting, and the outstanding disadvantages are that it cannot resist shock from heavy steamers and that the weight is so much in excess of steel.

Concrete Sinkers have to be occasionally used in and about ports and harbours for the attachment of moorings, and as concrete loses practically a half of its weight when submerged the blocks are oftentimes most unwieldy, and cast-iron sinkers have sometimes to be substituted in places where depth is a consideration.

Accompanying this article are photographs of a hollow sinker recently designed and made by Mr. H. Blake Thomas, M.I.Mech.E., of Holyhead, for use in an open port in the Mediterranean.

The construction is at first a buoy, then when scuttled becomes a sinker to which mooring may have been previously attached. This sinker was designed to overcome a difficulty created by the absence of lifting plant and to afford a weightier sinker than could be obtained by a solid concrete block of the same dimensions.

The sinker, with moorings and buoy attached, may be towed to the desired spot by a rowing boat and there scuttled without the aid of any further plant; the manholes to be subsequently broken open by a diver and the shell filled with scrap iron or old chain to any desired weight.

The following are the leading dimensions and weights :—

Diameter of sinker, 8 ft. 8 in.

Height of sinker, 4 ft. 8 in.

Thickness of walls, floor and roof, 3 in.

Draught (floating), 3 ft. 5 in.

Freeboard (floating), 1 ft. 3 in.

Gross weight, 5 tons 8 cwt.

Nett weight after scuttling, 2 tons 18 cwt.

Effective weight with chain ballast, 11 or 12 tons.

Number of watertight compartments, 8.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

AMERICAN SOCIETY OF CIVIL ENGINEERS. CONCRETE-LINED RESERVOIRS FOR OIL.

By E. D. COLE.

The following is an Abstract from a Paper read before the American Society of Civil Engineers, taken from the "Proceedings" of that Society, Vol. XLII., No. 6. This Paper describes the construction of concrete-lined earthen reservoirs used for storing crude petroleum in the oil fields of California, giving in detail the methods used to obtain safe and economical results, together with the total cost of storage per barrel of capacity. The method used in building up a compact and unyielding embankment as a backing for the concrete lining is given special attention. The losses of oil, due to evaporation and seepage, are given for both lined and unlined reservoirs, showing the economy of the concrete lining.

QUITE a number of large earthen oil-storage reservoirs have been constructed in the numerous oil fields of California, and the writer, being unable to find any literature covering this subject, and having been actively engaged for the past three years on construction of this class, thought that a brief description of the methods used and some unit costs would be of interest to engineers.

In general, these reservoirs are circular in plan, with earthen embankments built up from material excavated from within the reservoir, and range in capacity from 500,000 to 1,000,000 barrels. Some of the larger ones are oblong in plan, with circular ends connected by a rectangular section in the middle. In many cases dependence has been placed on the earth alone to prevent loss of oil by seepage, but it is now becoming general practice to line the inside slope and bottom with a thin layer of reinforced concrete as an additional safeguard against leakage. The reservoirs are covered with a wooden roof supported usually by 6 by 6-in. or 8 by 8-in. posts resting on wooden footings or concrete piers.

It is the purpose of this Paper to describe the construction of one of two 750,000 barrels reinforced concrete-lined reservoirs built by the writer for the Kern Trading and Oil Company, at Bakersfield, Cal., during the winter and spring of 1913-14. The general dimensions of the reservoirs are as follows:—

Inside diameter (bottom)	462 ft.
Inside diameter (top)	528 "
Depth	22 "
Width of top of embankment	11 "
Inside slope	1½ : 1
Outside slope	1½ : 1
Thickness of concrete lining (bottom)	3 in.
Thickness of concrete lining (top)	2½ "

The formation at the site was a light sandy clay, and this was easily handled by the Fresno and wheel scrapers used throughout the work. After the site was cleared of all brush and grass, the foundation under the embankment was thoroughly plowed and wet down before the fill was started. Wetting down the excavation material was

a help in several ways, as it not only made a more compact bank, but kept the dust down and made the earth ride better in the scrapers. This may seem inconsistent, inasmuch as the work was done during the rainy season, but can be readily understood on taking into consideration the fact that only six working days were lost during the winter on account of wet weather. The embankment was built up in thin layers, about 3 in. thick, laid parallel to the floor of the reservoir, and well compacted. In addition to the tramping of the scraper teams, the fill was compacted by two petrolithic road tampers which were driven continually around the top of the embankment.

To ensure a compact and uniform backing for the concrete lining on the inner slope below the natural ground surface, the excavation was started 2 ft. (measured normal to the slope) inside the inner line of slope stakes. This necessarily increased the quantity of excavation, and left the embankment short on the inner slope by this quantity. After the completion of the main portion of the embankment a lining of selected material, 3 ft. thick (measured normal to the slope), was built up against the inner slope, from sub-grade (1 ft. below floor grade) to the top of the finished fill. This extra foot of material was put on to insure a compact surface at the grade line of the inner slope, but was afterwards removed, as will be described later. The refill is an important part of the construction, because it would cut off any layers of sand or loose material that might be encountered in that portion of the inner slope which lies below the ground surface. On some previous work it was found necessary to excavate and refill portions of the natural embankment, below the ground surface, after the fill had been completed and trimmed to grade; this work caused considerable delay and added expense. One concrete-lined reservoir in this field partly failed, due to neglect of this part of the work, necessitating heavy expense in emptying the reservoir, besides the loss of a considerable quantity of oil and the cost of patching the lining.

On the completion of the main embankment and the refill, the excess material on the inner slope, which ranged in thickness from 1 ft. at the top to 2 ft. at the bottom, was trimmed off, leaving the slope smooth and true to grade. For this work of trimming the slope a novel and (the writer believes) original method was used. Grade stakes were set on radial lines, both at the top and inner toe of the slope, approximately every 10 ft. around the circumference of the reservoir. Men with mattocks and slope-level boards then dug narrow trenches, 1 ft. wide and true to grade, from the top grade stake to the stake at the toe of the slope. Then 2 by 4-in. timbers, 38 ft. long, each faced with a narrow strip of strap iron, were placed in the bottom of each trench to act as guides for a trimming machine which was used to finish that portion of the slope between the hand-dug trenches. Before using the planer, however, all excess material above the top of the 2 by 4-in. timbers was scraped off the slope with a specially made Mormon or Buck scraper, which was dragged up and down the slope, power being furnished by a double-drum hoisting engine at the centre of the reservoir. The back-up line from the engine passed through a 12-in. snatch-block supported at the top of the slope on a portable wooden truss designed for that purpose. This wooden truss was anchored against overturning, by two heavy chains fastened to iron stakes driven into the top of the embankment. As each succeeding section of the slope was finished, the wooden truss was moved along the top of the bank with a team of horses.

After the bulk of the material above the top of the 2 by 4-in. timbers had been removed, a slope trimming machine, designed and built by Mr. C. O. Zeller and the writer, was substituted for the Mormon scraper and used to plane off the remaining thin layer of earth and bring the slope to grade, or flush with the bottom of the guides. The trimming machine consists of a rectangular frame, 11 ft. long and 6 ft. wide, built up of 6-in. steel channels bolted together and carrying two cutting blades. The cutting blades are of $\frac{3}{4}$ by 12-in. flat rolled steel, and are set at an angle with the frame of 1 to $2\frac{1}{2}$. The blades are also set at a slight angle longitudinally with each other, and the cutting edge projects down 2 in. below the bottom of the frame. The planer is dragged back and forth on the slope until the ends of the frame ride on the top of the guides, and that particular section is shaved off flush with the bottom of the guides, or down to grade. In this way nearly nine-tenths of the slope were finished by machine and at one-half the cost of doing the work by hand.

The floor of the reservoir was excavated to a depth of 1 ft. below grade, and the

sub-grade was plowed and well rolled before commencing the refill upon it. It was then back-filled to grade, built up in thin layers, moistened and well rolled with petrolithic road tampers. Sand and gravel patches in the floor below sub-grade were excavated to a depth of 3 ft. below floor grade and then refilled with selected material. The floor was rolled smooth and hard to grade with a 5-ton road roller, a variation of only 0.05 ft. from true grade being allowed.

Excavations for the pier footings were made and the concrete piers for the roof supports were poured. Before pouring the concrete piers, however, and after the footings were dug, the floor reinforcing metal, which consisted of one layer of Clinton electrically welded fabric, made up of No. 6 wire in 6-in. square mesh, was laid out over the floor and tied together. The piers were poured through this metal, and later, when the floor was poured, this method left the reinforcing metal continuous throughout the floor and piers, without any patchwork.

The wooden roof was supported on 6 by 6-in. posts, approximately 18 ft. apart on centres, resting on concrete piers, and placed in concentric rings around the floor of the reservoir.

The floor and slopes of the reservoir were covered with a lining of concrete reinforced with a continuous layer of Clinton electrically welded fabric, made up of No. 6 wire in 6-in. square mesh. The thickness of the floor lining was 3 in., and that of the slope 2½ in. The concrete was mixed in a Ransome batch mixer, of 1 cu. yd. capacity, placed at the outside toe of the slope, and was delivered into a hopper at the toe of the slope inside the reservoir by a hoisting tower and 1 cu. yd. automatic dump bucket. From the hopper it was delivered into concrete buggies and wheeled to the spreading gang. In pouring the floor, a 1½-in. layer of concrete was first spread over a small section, and the reinforcing metal was then raised up through the concrete before the second layer was poured. After pouring the slab, the concrete was tamped with wide-faced wooden tampers, and troweled smooth and even. The construction joint at the end of a day's work was bevelled off, and, before placing concrete against it, on the following day, was swept clean of loose material and thoroughly grouted with a mortar of neat cement.

The concrete for the slope was delivered from the hoisting tower into small hoppers set on wagons and hauled with teams around the top of the embankment. From the wagons it was poured directly on the slope; no forms of any kind were used, as the concrete was mixed so that it would not run any more than enough to facilitate spreading. The slope lining was also put on in two layers, in order to ensure that the reinforcing metal would be placed in the middle of the slab. No trouble was experienced in pouring the slope without the use of forms, as the concrete was mixed fairly "stiff," and was prevented from "sluffing" by the reinforcing metal. In fact, the writer has just finished lining a reservoir with side slopes of 1 to 1, for the Union Oil Company, at San Luis Obispo, Cal., without the use of forms and with no appreciable difficulty. Each day's work was gone over on the day following and swept with a cement wash mixed in the proportion of 1 part cement to 1½ parts of water. This wash of grout was applied to fill up any small cracks which might appear while the concrete was taking its final set.

No expansion joints, either in the floor or slope, were put in on account of the comparatively heavy reinforcement of the slab and the low range in temperature to which the concrete would be subjected.

The concrete for the floor lining was mixed in the following proportions, based on a mechanical analysis of the aggregates:—

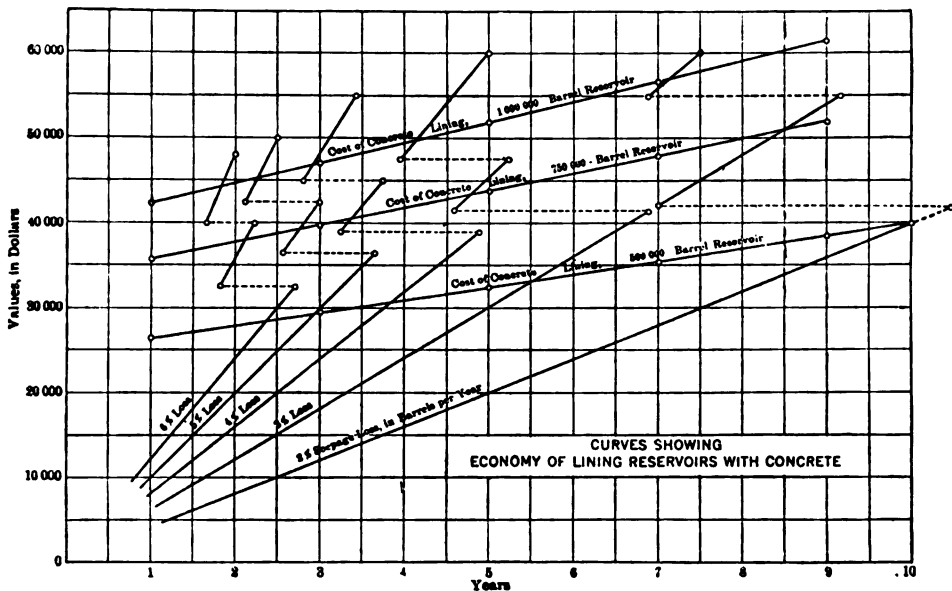
- 1 part cement.
- 3 parts sand.
- 1½ parts of ½-in. crushed rock.
- 2½ parts of 1-in. crushed rock.

This mixture worked well on the floor, but was found too rocky for the slope, as it was very difficult to tamp the concrete and prevent it from sluffing to the bottom of the slope. For this reason the 1-in. rock was eliminated, and the following mixture used:—

- 1 part cement.
- 2 parts sand.
- 2 parts ½-in. crushed rock.

The concrete crew, consisting of a foreman and thirty men, poured an average of 20,000 sq. ft. of lining per nine-hour day.

The losses of oil in unlined earthen reservoirs are due to both seepage and evaporation, and this, no doubt, is true to some extent in those that are lined with concrete. However, it seems reasonable to suppose that in the latter case the losses due to seepage are negligible, as concrete properly proportioned is impermeable to



oil of low gravity under heads of from 18 to 24 ft. This fact was proven to the satisfaction of the writer after an inspection of the condition of the concrete in a reservoir which had been standing full for two years. A scratch with a penknife removed the scum of oil from the surface of the lining and showed the concrete white and unattacked in a number of places examined. An additional test was made by suspending test slabs of concrete in oil to a depth of 20 ft.; after an immersion of six months no apparent penetration was noted where the flat surfaces of the slabs were troweled smooth and close-grained, but there were some signs of penetration on the corners and edges where the concrete was rough and small pieces of stone at the surface were exposed. This was hardly a fair test, as in placing a lining in a reservoir all the exposed surface is troweled smooth and an effort is made to embed all coarse material below the surface of the concrete. In the writer's opinion it is important that concrete used for this purpose should be made of fine material, and that no crushed rock or gravel in sizes larger than $\frac{3}{4}$ in. should be used for thin slab construction, on account of the difficulty in obtaining a smooth, close-grained finish with larger sizes. If this precaution is taken and the concrete is properly proportioned, it can be made impermeable to the higher grades of oil.

Average measurements of several concrete-lined reservoirs, taken during a period of one year, show a loss of approximately 1 per cent. of the volume per year, whereas in the case of the unlined storage the losses range from 1 to 7 per cent. per year. The wide variations in the latter are due to differences in the gravity of the oil and the varying character and compactness of the earthen lining. Taking into account the large quantities involved and the high value of the oil, it is evident that in many cases where the losses are large it would be economical to put in a concrete lining.

The curve shown above was plotted with the view of showing in any given case the number of years in which the seepage losses in an unlined earthen reservoir would pay for putting in a concrete lining. It is based on the following assumptions:—

Cost of lining 1,000,000-bbl. reservoir ...	\$0.04	per bbl.
" " 750,000- " " ...	'04½	" "
" " 500,000- " " ...	'05	" "
Value of oil ...	'40	" "
Interest computed at ...	6 p.c.	per annum.
Total loss of oil after concreting ...	1 p.c.	" "

In using this diagram it must be borne in mind that the evaporation loss is the same in the unlined as in the lined reservoir, and, for any specific case, 1 per cent. of the total loss of oil should be deducted. For example: A reservoir having a capacity of 750,000 barrels loses 5 per cent. of oil per year, 1 per cent. of which is due to evaporation and 4 per cent. to seepage. In this case, 4 per cent. loss, at a value of 40 cents per barrel, represents an actual cash loss of \$12,000 per year. By following up the 4 per cent. loss line on the diagram it will be seen that a concrete lining would pay for itself in about 3½ years.

In the computations for this diagram the actual capitalised value of the oil loss was not considered, as the aim was merely to show the length of time in which the actual loss in any specific instance would balance the expenditure necessary to line with concrete, and the accrued interest on this cash outlay.

Concrete-lined reservoirs of this type cost complete from 9½ to 10 cents per barrel of capacity, depending on the location and other governing conditions. This cost may be distributed approximately as follows:—

Cost of earthwork ...	\$0.03	per bbl. of capacity.
Cost of roof ...	'03	" "
Cost of concrete lining ...	'04	" "

The work was done by Mahoney Brothers, Allison and Cole, of San Francisco, with the writer acting as superintendent of construction. Mr. Earl Derby was resident engineer for the Kern Trading and Oil Company. Acknowledgment is made to Norman M. Halcombe, Assoc.M.Am.Soc.C.E., for help in preparing this paper.

MEMORANDA.

Law and Engineering.—At a meeting of the Society of Engineers (Incorporated), Mr. Sydney G. Turner, A.M.Inst.C.E., Barrister-at-Law, read a paper entitled "Law and Engineering—Some Points of Contact." The general object of the author was to show that in almost every branch of engineering practice the provinces of the engineer and the lawyer overlap, and to advocate the establishment of some periodical meeting, open to the members of both professions, at which subjects of common interest could be discussed. For this purpose a brief survey was made of the principal branches of engineering, and in regard to each illustrations were given of such subjects by way of suggestion. The subjects of the giving of expert evidence, the duties of the engineer when acting as arbitrator, and in connection with the preparation of contracts and certificates were first dealt with generally. The author then dealt with municipal engineering, and pointed out that municipal engineers have a special interest in law, inasmuch as their work consists almost entirely in the administration of Acts of Parliament. The problems raised for the engineer and the lawyer in connection with modern traffic conditions and highway construction were next indicated; while the latter part of the paper dealt in turn with points concerning gas engineering, water engineering, and electrical engineering respectively. In the section dealing with municipal engineering, the special topics selected were sewerage and drainage, and the execution of works of private street improvement. In regard to the former, the decision of the House of Lords in the case of *The Wood Green Urban District Council v. Joseph* was discussed; and in regard to the latter the author dealt more particularly with the duties of the engineer in connection with the apportionment. In the section dealing with water engineering, the question of underground supplies was suggested as a subject for discussion. In conclusion, the author expressed the hope that in the course of the discussion opinions might be offered as to the desirability of establishing some such meeting as was advocated in the paper.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE IN THE ELECTRIFICATION OF THE LONDON & SOUTH-WESTERN RAILWAY (Suburban Lines).

A GREAT deal of important work is being carried out by the London and South-Western Railway in connection with the electrification of a portion of their system, and reinforced concrete plays an important part in this work. The system employed is the Mouchel-Hennebique, executed in accordance with details prepared by Messrs. Mouchel and Partners, of Victoria Street, Westminster, for the Electrical Engineer of the railway.

An interesting example of this work is to be seen in the viaduct constructed at Durnsford Road Power House, Wimbledon, this being about 550 ft. long and designed for carrying coal wagons to the overhead bunkers in the boiler-house. This viaduct has a varying height owing to the gradient required to bring the rails up to the required



Fig. 1. Viaduct at Durnsford Road Power House.

REINFORCED CONCRETE IN THE ELECTRIFICATION OF THE LONDON AND SOUTH-WESTERN RAILWAY.

level, and the over-all width is 19 ft. The portion adjoining the boiler house, for a length of 50 ft., is constructed with steel girders, and this is followed by 100 ft. run of viaduct of reinforced concrete formed with flooring 6 in. thick. The remaining 400 ft. is constructed with an open deck, and at the lower end of this portion where the viaduct ends the rails are carried on an embankment with a reinforced concrete retaining wall next to the viaduct, to give a vertical face and provide bearing for the viaduct beams.

A general view of the work can be seen in the photograph in Fig. 1, and the trestle

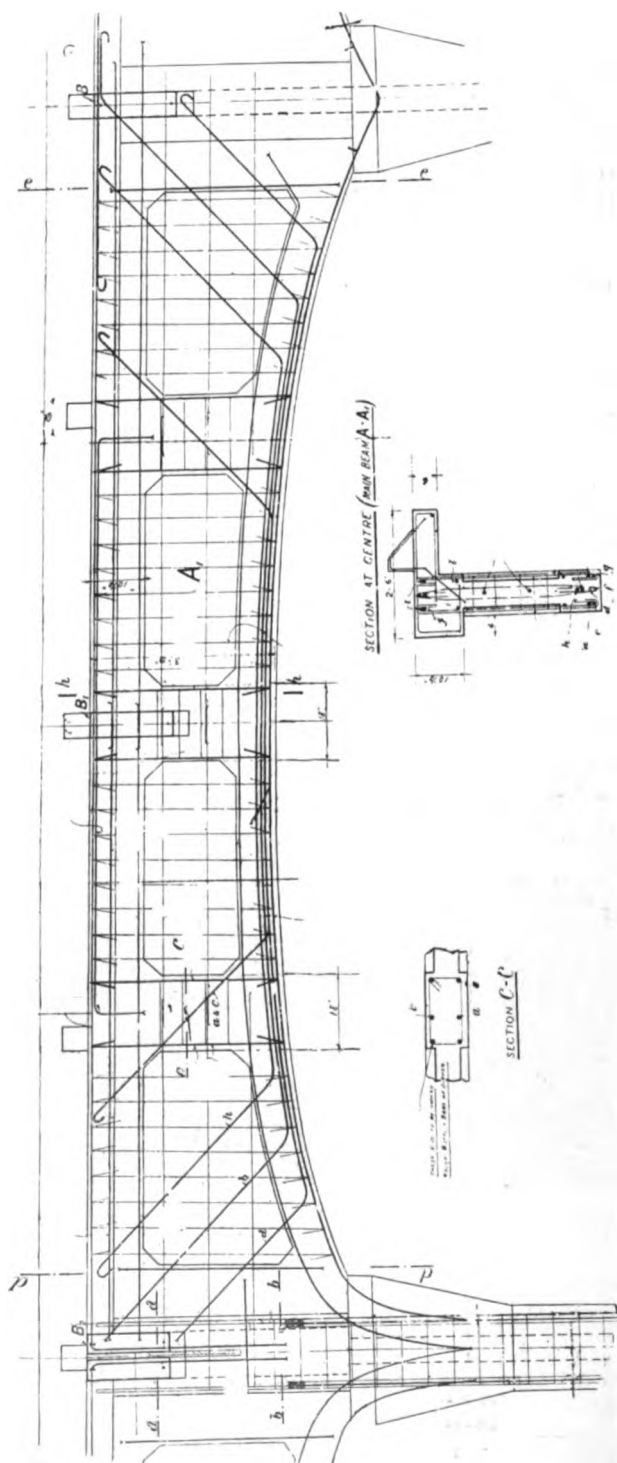


Fig. 2. Elevation of Girder of Viaduct.
REINFORCED CONCRETE IN THE ELECTRIFICATION OF THE LONDON AND SOUTH WESTERN RAILWAY.

supports which carry the decking are spaced at 25 ft. centres. The legs are splayed outwards at the bottom and ties are introduced below the ground level to prevent any spreading tendency. The load on the soil was taken at 4 tons per sq. ft., and the superload on the solid decking was calculated at 5 cwts. per sq. ft., while the maximum axle load diagram shows three loads of 12 tons at 6 ft. 10 in. centres, and two loads of $13\frac{1}{2}$ tons each at 8 ft. centres.

The detail illustrated in Fig. 2 shows one of the main girders which are immediately under the rails and above the legs. These girders are placed 5 ft. apart, and they are of arched type, with a curved soffit, and the web is reduced in thickness by the formation of panels on both sides, cut up into four bays by vertical stiffening portions. The top flange has a thickness of $12\frac{1}{2}$ in. on the inner side of the web, and a thickness of 6 in. on the outer side, with a total width of 2 ft. 6 in. The bottom flange is 10 in. square, and the total depth of the beam varies from 3 ft. 9 in. at the centre to 5 ft. 8 in. at the support. Six main rods are placed in the upper surface of the top flange and eight rods in the

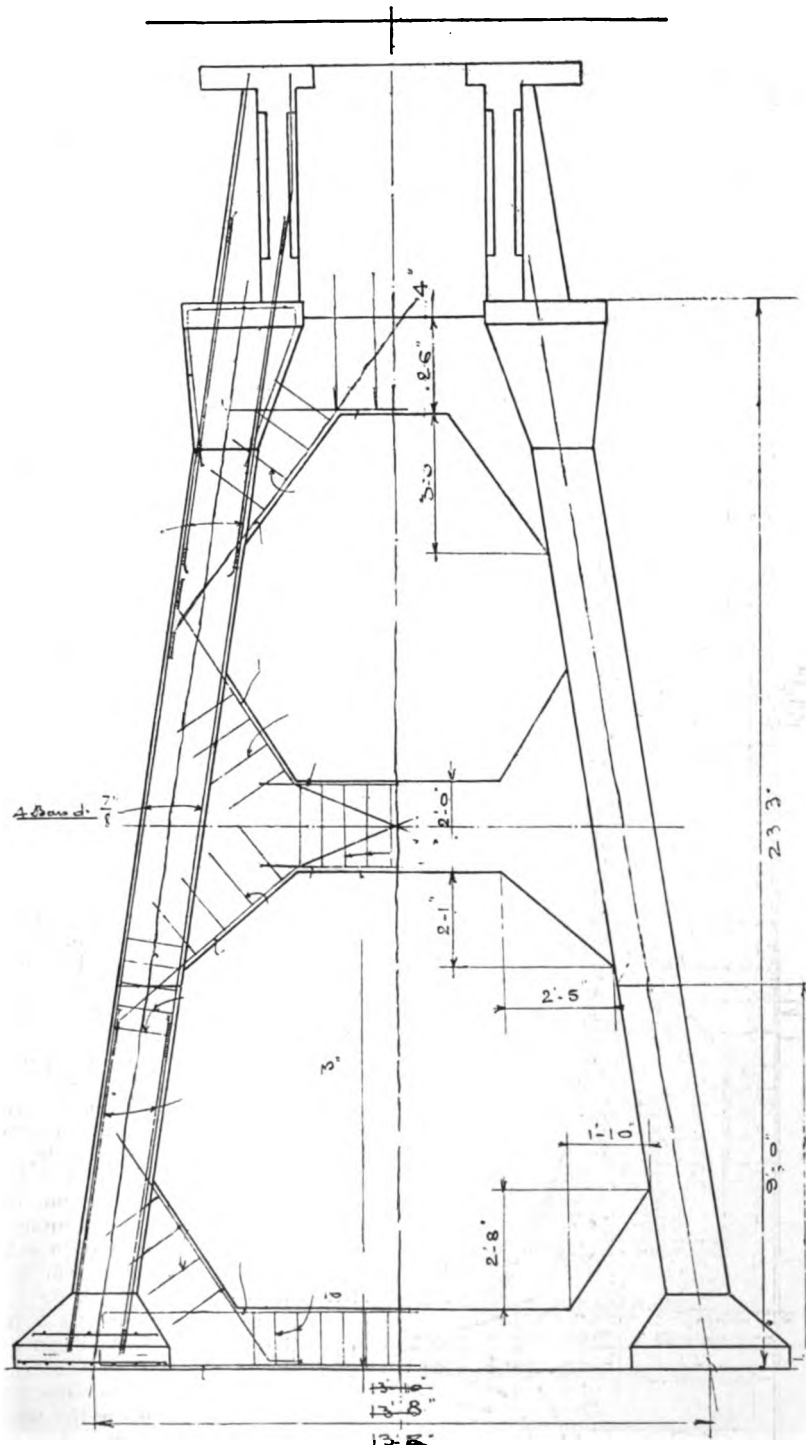


Fig. 3. Cross Elevation of Columns of Viaduct.

REINFORCED CONCRETE IN THE ELECTRIFICATION OF THE LONDON AND SOUTH-WESTERN RAILWAY.

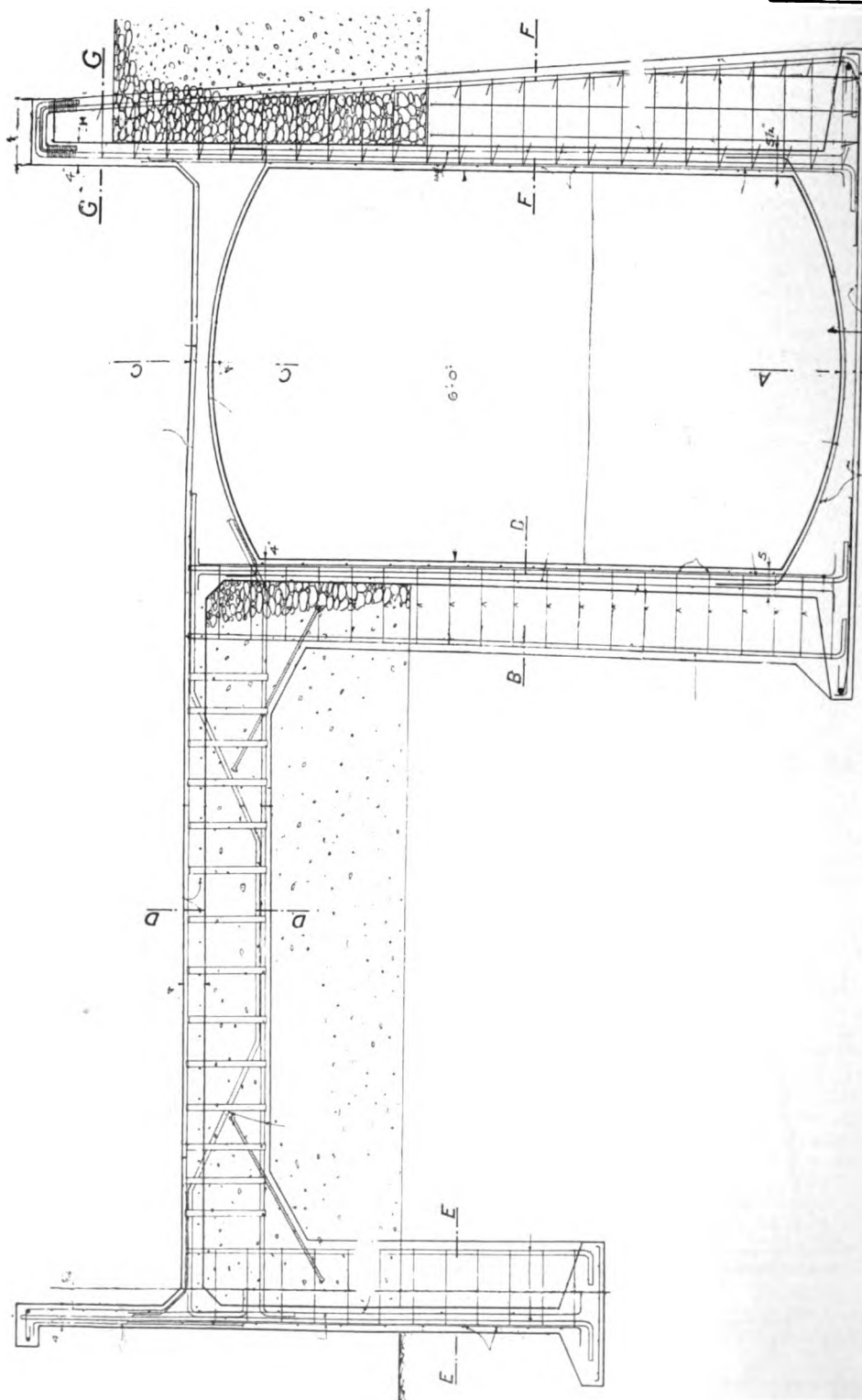


Fig. 4. Cross-Section through Reinforced Concrete Aqueduct.
REINFORCED CONCRETE IN THE ELEVATION OF THE LONDON AND SOUTH WESTERN RAILWAY.

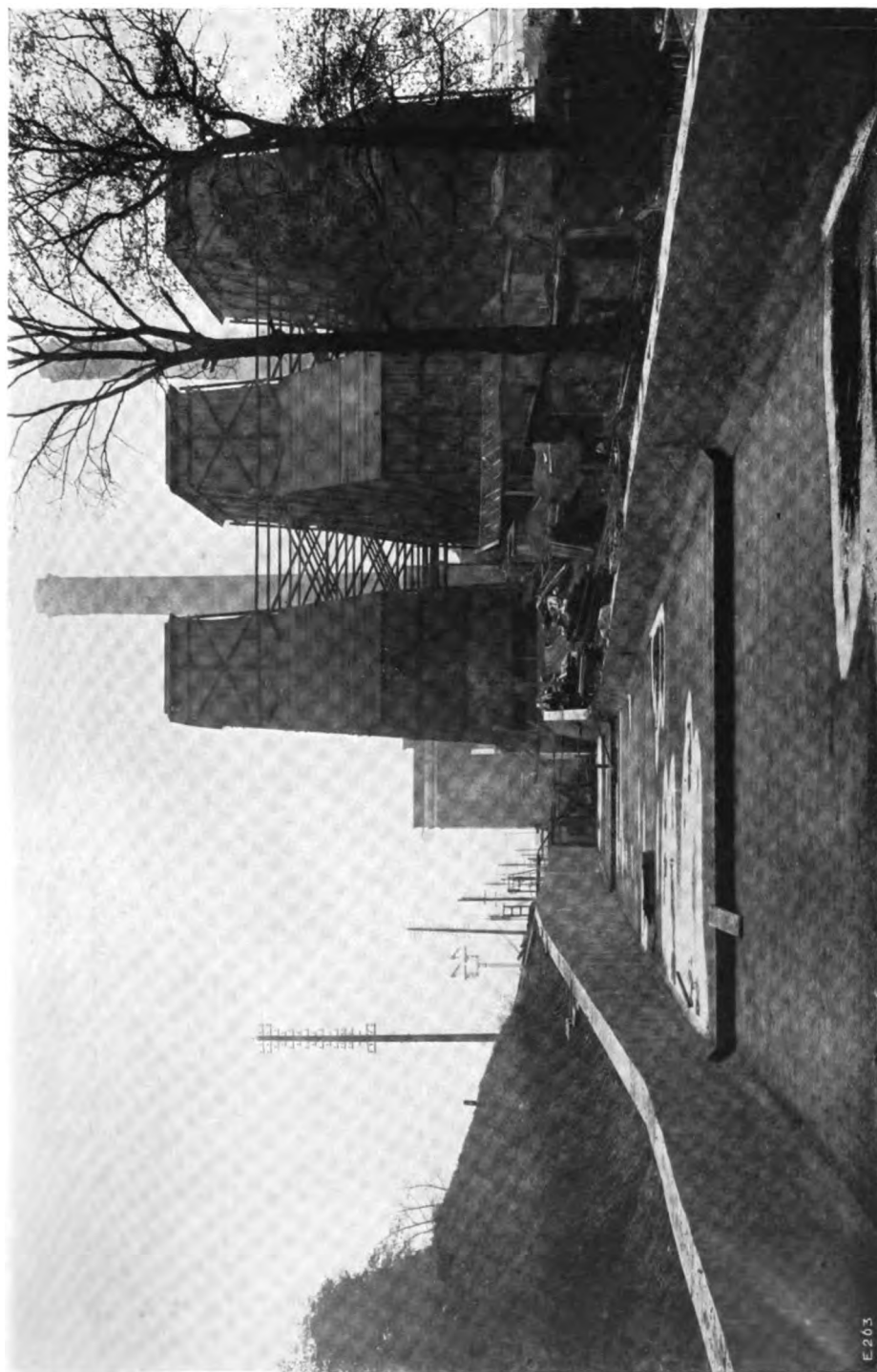


Fig. 5 View of Aqueduct.
REINFORCED CONCRETE IN THE ELECTRIFICATION OF THE LONDON AND SOUTH-WESTERN RAILWAY.

Photograph by Tella Camera Co., London, W.C.1

E 203

lower flange, while additional longitudinal rods are provided at the corners nearest the web and in the web itself, with stirrups and binding as shown on the drawing. A section of the stiffener is also given in the illustrations, and it will be seen that this is reinforced with six vertical rods. The sides of the viaduct outside the line of

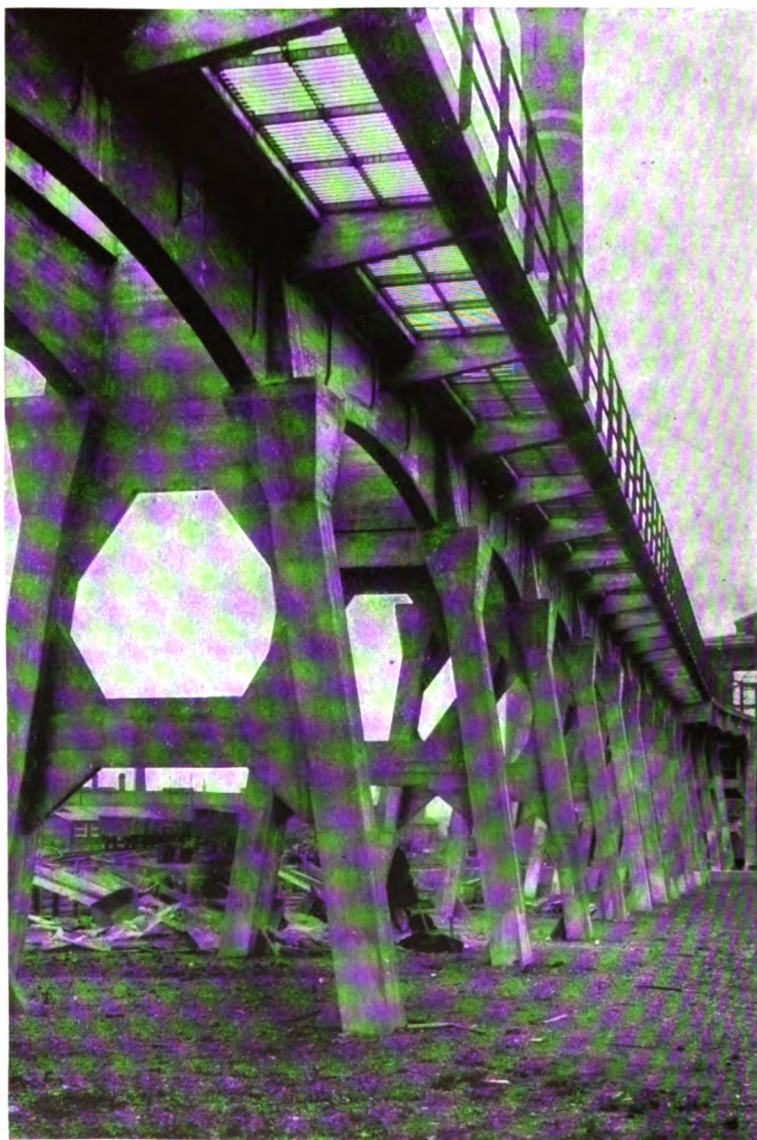


FIG. 6. View Showing Viaduct with Open Deck.

REINFORCED CONCRETE IN THE ELECTRIFICATION OF THE LONDON AND SOUTH-WESTERN RAILWAY.

rails is carried by cantilevers, which have a maximum depth of 2 ft. next the trestle, reducing to 12 in. at the outer end, one pair of cantilevers being placed at each trestle and one pair in the centre of each span. These cantilevers support continuous tee-beams which form the outer edges of the viaduct, these beams having a table 1 ft. 9 in.

wide and a 4-in. rib with a depth of 1 ft. 3 in., and three main longitudinal bars are introduced as reinforcement. A typical detail of a trestle is illustrated in *Fig. 3*, this consisting of two inclined legs, each 20 in. by 16 in., with horizontal ties at the head, foot, and centre of the height. The top tie is 2 ft. 6 in. deep, the centre one 2 ft. deep, and the bottom one 16 in. deep, the thickness employed in each case being 6 in., and the splayed haunches being formed at the junctions with the legs as shown. Each of these ties is reinforced with one rod in the upper and lower surfaces, and the legs are reinforced with four rods, while the latter have splayed bases 3 ft. 6 in. wide and 7 ft. long to distribute the load over the requisite area of soil.

The second example of reinforced concrete is that illustrated in *Fig. 4*, this showing the section of an aqueduct constructed alongside the Railway for carrying water from the River Wandle to the Condensing Plant in the Power House and back to the river again. This has a total length of over 900 ft., and the drawing given is a typical detail of the work. A retaining wall is constructed next the embankment, with panels $5\frac{1}{2}$ in. thick, with reinforcing rods in both surfaces and directions, and counterforts having a projection of 12 in. and a width of 6 in. at intervals of 7 ft. 6 in. These counterforts are splayed outwards towards the bottom to give a total width of 2 ft., and the foundation is formed by a continuation of the concrete forming the bottom of the waterway. The latter is arched at both top and bottom with reinforced concrete, having a minimum thickness of 4 in., and the width between the side walls is 6 ft. Over the aqueduct large tanks, 18 ft. wide, are constructed with 4 in. of reinforced concrete at the bottom carried on 12 in. by 6 in. reinforced concrete beams at 7 ft. 6 in. centres. The outer walls of the tanks, together with the supports under the outer ends of these beams and the general design of the work, can be followed upon referring to the detail previously mentioned.

These two examples are interesting types of reinforced concrete well designed and applied, and the photographic views show the general aspect of the finished work.

The contractors for the viaduct were Messrs. Holloway Bros. (London), Ltd., and for the aqueduct Messrs. J. Garrett and Son.

Mr. Herbert Jones, Electrical Engineer to the London and South-Western Railway, was responsible for this work, which was carried out under his supervision.



Fig. 7. Viaduct at Durnsford Road Power House.

REINFORCED CONCRETE IN THE ELECTRIFICATION OF THE LONDON AND SOUTH-WESTERN RAILWAY.

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

The Strength of Materials. By Ewart S. Andrews, B.Sc.

London: Chapman and Hall. 604 pp. + x. Price 10/6 net.

Contents.—Stress, Strain and Elasticity:—The Behaviour of Various Materials under Test—Repetition of Stresses—Working Stresses—Riveted Joints; Thin Pipes—Bending Moments and Shearing Forces on Beams—Geometrical Properties of Sections—Stresses in Beams—Deflection of Beams—Columns, Stanchions, and Struts—Torsion and Twisting of Shafts—Springs—The Testing of Materials—Fixed and Continuous Beams—Distribution of Shearing Stresses in Beams—Flat Plates and Slabs—Thick Pipes—Curved Beams—Rotating Drums, Disks, and Shafts—Exercises.

An idea of the extensive matter contained in this book can be gathered from the contents given above, and when it is stated that the author has spared no trouble in dealing with any of the sections enumerated it will be understood that it is a complete treatise of the subject. In a volume of this kind a great deal of its value is dependent on the manner in which the elementary principles are explained and the construction of the various formulæ expressed, because it is essential for the reader thoroughly to understand the reasoning which leads up to a rational formula if intelligent application is to result. The author has fully appreciated this fact, and students will find no difficulty in following the theory with a proper understanding.

There are many sections which are of special interest, and amongst these may be mentioned those dealing with columns, stanchions, and struts, and fixed and continuous beams. These two matters are of particular interest, and they are dealt with in an able manner, which leaves nothing to be desired. The deflection of beams is also well treated, and the chapter under this heading should be of much assistance to the average student, who usually considers it a difficult subject. Some of the matter contained in the volume will not be generally required in ordinary building work, but a study of the theory put forward

will assist in developing the reader's reasoning powers, and in any case it makes the book more useful as a work of reference. Although the book is specially intended for students, the author has endeavoured to present the subject in sufficiently practical form to be of greater assistance in practical design than is the case with an ordinary text-book, and in this respect it is successful. Numerous exercises and tables are given, together with many illustrations. We can thoroughly recommend the volume as one possessing considerable merit.

Wind Stresses in the Steel Frames of Office Buildings. By W. M. Wilson and G. A. Maney. Bulletin No. 80, University of Illinois.

Chapman and Hall, Ltd., 11 Henrietta Street, Covent Garden, London, W.C.

The present bulletin deals with a subject which has presented considerable difficulty in the design of steel frames for buildings, and which, as far as we are aware, has never been thoroughly investigated. The bulletin investigates the stresses by deriving fundamental formulæ from a consideration of the slopes of the deflected form of the beams and columns; the result for any given frame is, of course, a very complicated series of summations. A proposed approximate method is then given, which is derived by making in the main theory certain additional assumptions which are found to have little numerical effects upon the results. Curves are given in the bulletin for enabling this approximate method to be used, and, although the method is longer than the approximate methods usually adopted, it should not take very long when it has become familiar by use.

A numerical example is worked out fully by the accurate method, the proposed approximate method, and by four rough approximate methods in common use, commonly known as Fleming's methods and Smith's method. In this case it is found that the best rough approximate methods are those which assume the points of contraflexure of both beams and columns at their mid-points.

The bulletin contains also the results of tests upon a very small celluloid model,

and the conclusion given is that the results of the experiments agree very well with the calculated results.

The experiments appear to be the weakest part of the investigation; when it is remembered that the bending moments are computed from the changes in slope, and that these changes are very small, very reliable numerical results cannot be expected with a model 14 in. high and $10\frac{1}{2}$ in. wide made with celluloid $\frac{1}{8}$ in. thick.

The principal value of the bulletin—and the value is great—is the analysis of the stresses; we hope that at some future date an experimental investigation upon a more ambitious scale may be attempted, so that we may see the accuracy which we may

expect in the use of the method proposed. We should never lose sight of the fact that, in comparing the relative accuracies of a frankly approximate method of stress calculation with one based upon more reliable assumptions, we should compare each with the actual stresses, and not compare the stresses by the more rigid analysis with that obtained by the approximate method. In making this statement we are aware of the very great difficulty in the way of accurate experiment; many engineers who clamour for all theory to be swept away by experimental results do not appreciate fully the difficulties involved in the conduction of the experiments nor the skill and experience required in interpreting their results.

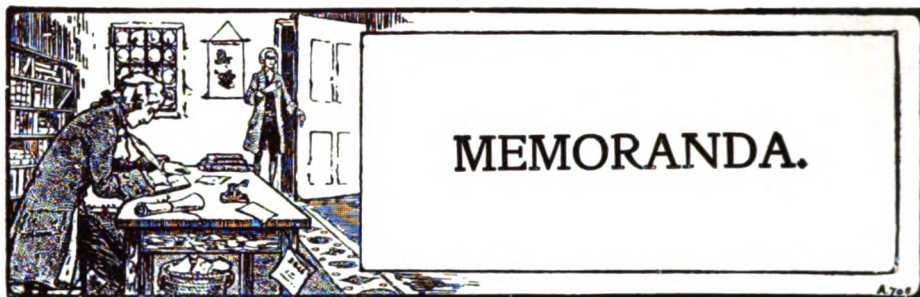
MEMORANDA.

Concrete Scow has Withstood 4½ Years' Hard Service.—Apropos of the experiences with concrete barges on the Panama Canal, detailed in the *Engineering Record*, it is worth noting that a concrete barge has been in service on the Welland Canal for four and a half years, and during that time has been subjected to severe service. It is used in canal maintenance work, and at times riprap, in loading, has been dropped on its deck from a height of 12 ft.

The scow, which was built in November, 1910, is 80 ft. long, of 24 ft. beam, and has sides 7 ft. high. The hull is divided into eight compartments by one longitudinal and three cross bulkheads. There are 2 by $2\frac{1}{2}$ ft. openings through the cross bulkheads. The deck, bottom, sides and bulkheads are $2\frac{1}{2}$ in. thick, reinforced in both directions by $\frac{1}{4}$ -in. round rods on 2-in. centres. Beams 6 in. square on bulkhead lines, reinforced with heavier steel, as required, and braced by reinforced concrete posts, care for the principal stresses. The concrete is a 1 : 4 mix, using small gravel.

The boat was designed and built under the supervision of J. L. Weller, then superintending engineer of the Welland Canal, and now engineer-in-charge, Welland Ship Canal.

Paving Streets with Concrete.—An innovation is to be tried by the city engineering department in the paving of a street in Schenectady, New York. This is to be paved with a 7-in. deep single course layer of concrete, and will be the first real concrete street in the city. The stretch to be paved is about 450 ft. long by 24 ft. wide. The novel feature, that has not been tried elsewhere to the knowledge of the department, will be expansion joints alongside of the kerb and every 20 ft., running across the street diagonally at an angle of sixty degrees. That is, the pavement will be divided up into diamond-shaped blocks each running the entire width of the street and along the kerb for 20 ft., but separated from the kerb and the adjoining blocks by a space varying from $\frac{1}{4}$ in. at the kerb to $\frac{3}{4}$ in. between blocks and filled with tar asphaltum or similar material. The reason for having these joints run diagonally is to minimise the shock of the concussion of wagon wheels, as by this arrangement only one wheel would strike the joint at a time instead of both forward and back wheels together, as would be the case were the joints or cracks perpendicular to the kerbstone. The kerbstone will be of concrete also, and will be 6 in. wide by 24 in. deep.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Long Concrete Pier at Tiree.—A new concrete breakwater has just been completed at Gott Bay, Tiree, an island in the Hebrides on the West Coast of Scotland. In length the pier is 571 ft., sheltered by a parapet wall, and followed by an open viaduct 250 ft. long by 12 ft. wide. The breakwater is of concrete, and the work of construction was rendered very severe by the exposed position of that part of the island. Heavy storms had frequently to be faced. The reinforced concrete pier for the use of steamers was constructed in accordance with the Mouchel-Hennebique system from drawings prepared by Messrs. L. G. Mouchel and Partners, Ltd. This method is similar to that carried out by that firm elsewhere, with the addition of a very strong viaduct.

The foundation consists of reinforced concrete piles driven deep into the firm sands of Gott Bay, and the superstructure of columns in continuation of the piles, the whole being braced by horizontal and diagonal members as well as by the longitudinal and transverse beams of the decking, and, finally, by the continuous decking slab. All the reinforced concrete is in accordance with standard Hennebique practice, and the work is monolithic and jointless throughout. The open viaduct following the breakwater comprises four steel girder spans carried on massive concrete piers, founded on solid rock 5 ft. to 6 ft. below the bed of the sea. The remainder of the viaduct consists of reinforced concrete girder spans carried on piers formed of Hennebique patent piles and bracing. At the end of the viaduct is the pier head with a frontage of 150 ft. and a width of 25 ft., constructed entirely in reinforced concrete.

Gott Bay pier is the longest pier on the West Coast of Scotland. The total length is 971 ft., and its projection into the sea is 846 ft. An excellent slip has also been constructed on the harbour side of the breakwater for use in times of heavy ground swell. The viaduct has a railing on each side, and carries a tramway for trolleys to convey goods to a warehouse at the shore end of the breakwater. Cargo sheds, offices and waiting rooms are also on part of the decking at the pier head. Messrs. Kinnear, Moodie and Co., Glasgow, were the contractors, and experienced great difficulties on account of wild seas in this very exposed part of the Scottish coast facing the Atlantic. Mr. G. Woulfe Brennan planned all the works.—*Contract Journal*.

Port of London Authority—New Cold Stores.—Owing to the growth of the refrigerating trade, ever-increasing attention is being paid by our large ports to the question of adequate cold storage. The Port of London Authority already have considerable accommodation, and when their new works under construction are completed at the Royal Albert Docks the accommodation will be increased from 843,000 carcases to 1,350,000. The new cold store is a large and imposing building of reinforced concrete, 300 ft. long, 100 ft. wide, and over 70 ft. high, and has six floors. Five floors will be devoted to storage purposes. The store is so situated that produce can be easily conveyed from the ship to the building by means of an electrically-driven endless conveyor. The works at present in hand only comprise two-thirds of the scheme in contemplation.

Reinforced Concrete Flagpole at Panama.—Mr. G. C. Dobson, Assistant Engineer, Permanent Building Division, Panama Canal, gives the following particulars in *Engineering News*:—A reinforced-concrete flagpole erected on Administration

Hill at Balboa Heights, Canal Zone, presents a unique piece of construction. It stands on the principal axis of the Administration Building and 59 ft. in front of it. Its position at the head of a spacious flight of steps from the terrace to the main townsite gives it added prominence.

The pole has a clear height of 70 ft., a ground penetration of 12 ft., and rests on natural soil. It is sixteen-sided and has a diameter of $8\frac{1}{2}$ in. at the top and 17 in. where it joins the ornamental base. It is reinforced longitudinally by eight square bars, which are $1\frac{1}{2}$ in. in thickness at the base, graduate down to $\frac{3}{4}$ in. at the top, and are welded so as to form continuous bars. The welds are reinforced and staggered. The pole is also reinforced by hoops of $\frac{3}{8}$ -in. square bars spaced every 4 in.

The pole was precast in position for erection in wood forms, which had to be constantly adjusted to keep the pole in alignment, due to the settling of the fill on which the forms were set up and to blasting in the neighbourhood. The concrete was a 1:1:2 mixture. Eighteen days after pouring it was erected. Stresses were developed during erection much greater than would be caused by a hurricane of 100 mi. per hour.

The lower 10 ft. of the pole consists of a spreading octagonal base of flutes and plain mouldings, with a seat and platform at the bottom. These members were all precast and set up after the pole was erected. They are about 3 in. thick and heavily reinforced with mesh. The pole itself received a thin coat of stucco after erection to true up the arrises and to remove all form marks.

To the top of the pole is secured a very heavy cast-brass pulleyless block, through a groove in which slides the flag halyard, and through the centre a 1-in. brass rod projects 1 ft., carrying a 12-in. copper ball, double gold leafed. The centre of this ball is about 167 ft. above sea level. At the base the halyard passes through the cap moulding and is secured to a cast-brass cleat directly below it.

The pole was designed and erected by the Permanent Building Division, Supply Department, Panama Canal. The erection was done under the direct supervision of the Division Engineer, Frank Holmes. The architectural design was handled by S. M. Hitt, Architect of the Panama Canal, and the engineering design by Mr. G. C. Dobson.

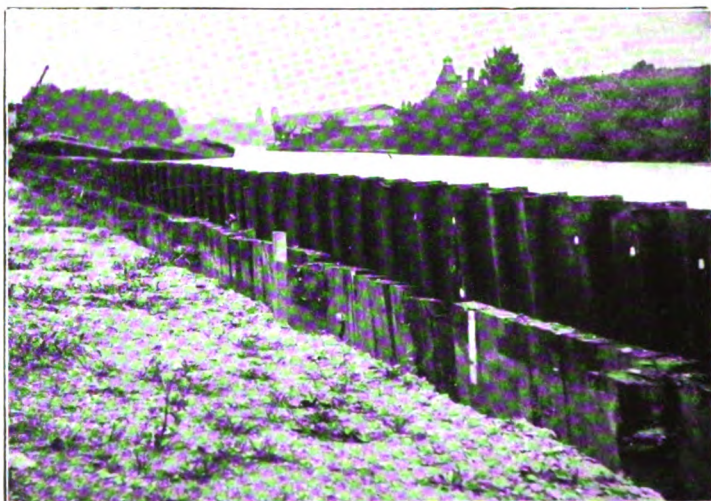
Erecting Gas Engines.—In an article on this subject in *The Ironmonger*, the author makes the following remarks with regard to material for foundations:—

Opinions differ not a little as to the most suitable proportions of the materials used in a concrete engine bed. The writer's view, based on a quarter of a century's experience in the fixing and erecting of medium and heavy engines, leans towards the best quality concrete. Often it pays, because, if the customer is in a hurry, days may be saved owing to the fact that the better the quality the quicker and harder a concrete sets. He can strongly recommend the following proportions: Broken rubble (stones or bricks), 4; clean river sand, limestone or granite chippings, 4; cement, 1. The last mentioned must be quite fresh if the best result is wanted.

Holding-down bolts should not be arranged with their heads below the concrete foundation, but should be embedded therein. It is usual to lay into the excavation a thickness of rough concrete ranging from 6 in. in the case of small engines to 18 in. in the case of large sizes; upon this first layer the bolts are set. They should in all cases have square necks, and be provided with washer plates from 3 to 12 in. square, and at least $\frac{1}{2}$ in. in thickness. It is no advantage to tie two bolts together with one long plate unless through any cause one or both of the bolts have an insufficient amount of concrete to support and hold it on its outer side.

Setting the bolts in position requires great care, for upon this depends the accurate setting of the engine. The best plan to ensure the necessary degree of exactitude is to construct a wood template of the engine bed, with holes set out in it to correspond with those in the bed. This may be made of 1-in. board nailed or screwed together, and if necessary tied diagonally to ensure stiffness. A centre line marked across it to correspond with a line taken through the crank-shaft materially facilitates setting. Next a set of light wood boxes to encase the bolts must be provided, each with an adequate clearance of the bolt in the foundation. If these boxes are made of $\frac{3}{4}$ -in. timber, very lightly tacked together, and well tapered, their breaking up and extrac-

The BRITISH STEEL PILING CO



CONSERVING RIVER BANK.

Above photograph shows our "UNIVERSAL JOIST" Steel Sheet PILING as driven by means of one of our No. 6 McKIERNAN-TERRY Double-Acting Steam or Air HAMMERS, by the Engineer to the Metropolitan Water Board, in front of old timber camp sheeting near Hampton Court, to conserve the bank of the River Thames against the eroding action of the current of the river.

Every description of PILE DRIVING PLANTS, AND both our types of STEEL SHEET PILING, are available FOR SALE OR HIRE.

WEIGHTS OF PILING:—

"SIMPLEX"	-	22 to 27 lbs. per sq. ft.
"UNIVERSAL JOIST"	-	from 43 lbs. "

DOCK HOUSE, Billiter Street, LONDON, E.C.

tion are rendered easy. In length they should be such that they will stand, say, 2 or 3 in. above the finished foundation, a plan which allows of their being gripped for pulling out. The boxes should have a clearance in them at their small end of at least two bolt diameters. They are placed under the bolts, small end down, and then the template is put on, with such washers as may be necessary to make up the bolt length.

The template has next to be packed up above floor level to the desired height, care being taken to allow for the thickness of the engine framing. The template in position will, of course, correspond with the square lines set out upon the floor. Everything is now ready for proceeding with the concrete bed. In filling this in start with the coarsest concrete, ramming it well down after each "gauge," and taking care that the bolt ends and plates are well packed, and that the bolts and boxes are not disturbed from their vertical positions.

As the bed nears completion a finer grade material should be used, but it is not wise to finish with too fine or too smooth a face; in fact, the rougher in reason the better, for such roughness provides an effective key for the final floating coat when the engine is placed in position. It is also advisable to arrange for the cylinder end of the bed to be slightly higher than the crank-shaft end; this gradient need not exceed a $\frac{1}{4}$ in. in every 3 ft. This dodge prevents oil draining back into the cylinder end. Wheel races should be formed in the bed with segmental templates of wood made to the desired curve. They are best set in position as the bed proceeds. Above floor level the concrete is kept in place by fixing boards on the edges to form a continuation of the size of the excavation; temporarily nailing at the corners suffices as a general rule.

A Concrete Porch at Elkhart, Indiana.—The accompanying illustration shows an artistic concrete porch at Elkhart, Indiana, at the home of Mr. H. W. Fisher. The special architectural feature of this porch is in the columns and balusters. The concrete columns are 8 in. by 8 in. square and fluted with neat beaded bands. The concrete



A CONCRETE PORCH AT ELKHART, IND.

balusters are 3 by 6 by 16, fluted and beaded to harmonise. This concrete verandah is 8 ft. wide and 20 ft. long, and has been built by the Artisan Cement Products Works from moulds which they have prepared at the Elkhart plant.

Moving Pictures showing Concrete Construction.—The Journal of the Western Society of Engineers, U.S.A., contains an interesting account of the presentation of moving pictures dealing with various forms of concrete construction. The films were furnished by the Universal Portland Cement Co., U.S.A. One of the

reels showed methods of road construction for city pavements and cross-country highways. Specially interesting were the pictures showing the methods of handling material.

Another reel of film illustrated methods of concrete structures on the farm, and included silo construction, fence posts, the application of the concrete tile, etc.

New Essex Bridge.—A new county bridge at Ferry Lane, Walthamstow, to take the place of the old timber structure, has been completed and opened for the Essex County Council. The span of the bridge is composed of open hearth acid steel, there being nine arched ribs 1 ft. 9 in. deep and 60 ft. long. The ends of each rib are fixed to steel girders buried in the concrete of the abutments. The abutments, which are 20 ft. wide at the base and fixed on a grid of 12-in. square timber piles 20 ft. long, are composed of Portland cement concrete faced with brickwork. The cost of the bridge was £8,300.

A Canadian Engineering Operation.—The seventh annual report of the Ontario Hydro-Electric Commission describes, amongst other things, how the spans across the Welland Canal were lengthened from 407 to 532 ft. To do this, one standard tower was removed from the transmission line and one anchor tower was moved 45 ft., and the two high towers supporting the canal crossing span were moved, one 63 ft. and the other 62 ft. The important part of the work was the moving of the two latter towers. Each weighed 25 tons and was supported on a heavy reinforced concrete base. The over-all length was 168 ft. The two towers were moved standing, being pulled along timber skidways to new concrete foundations.

TRADE NOTES.

Lantern Slides for Lectures.—Messrs. Ed. Bennis and Co., Ltd., inform us that they have a large number of lantern slides dealing with the development and present practice in connection with automatic stokers, coal elevators, and conveyors, etc., which they will be pleased to loan to any responsible engineer for lecture purposes. A list of the slides and particulars of their subject matter can be obtained on request.

Application for slides should be made as far in advance of the lecture date as possible, and should be addressed to Ed. Bennis and Co., Ltd., 28, Victoria Street, London, S.W.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

The VICTORIA is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

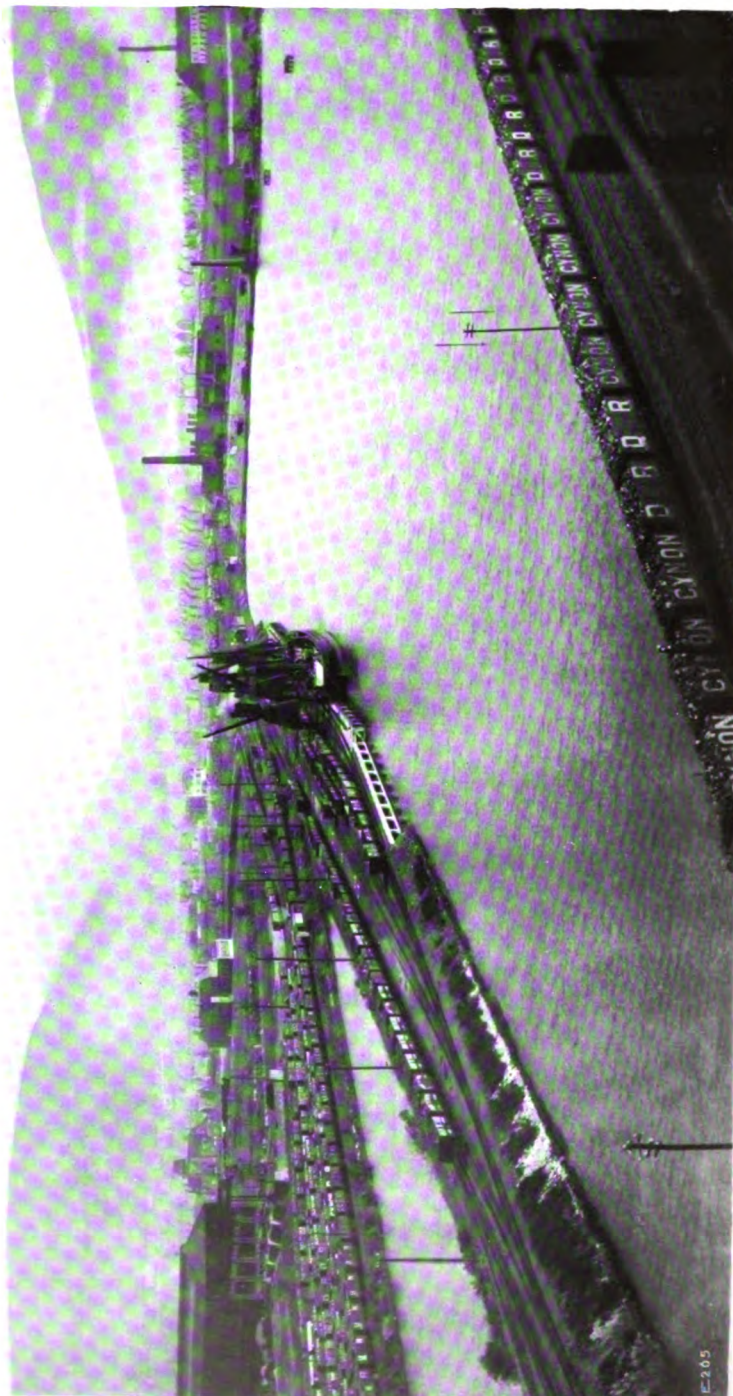
is built to last

WRITE FOR CATALOGUE No. 29.

THE

T. L. SMITH Co.
13, Victoria Street, S.W.

A
E



General View of the Wharf and Sidings.
NEW REINFORCED CONCRETE WHARF AND JETTIES FOR THE PORT TALBOT RAILWAY AND DOCKS CO.
(For Description see p. 601.)

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume X., No. 12.

LONDON, DECEMBER, 1915.

EDITORIAL NOTES.

BRITISH STANDARD SPECIFICATION FOR PORTLAND CEMENT.

THE following are some observations with regard to the changes made in the above specification which appear in the fourth issue, published in March last, affecting the tests for determining quality.

Fineness.—In the 3rd edition the diameter of the wire of the 180 mesh sieving was required to be 0.002 inch. In the new edition, however, this has been reduced to 0.0018 inch. This reduction, of course, has the effect of slightly increasing the size of the holes, and therefore reduces the amount of residue which would be left on the sieve by a given cement. This slight reduction in the stringency of the sieve, however, is much more than counter-balanced by the reduction of the amount of residue permissible from 18 per cent. to 14 per cent. This drastic reduction is undoubtedly to the advantage of the consumer, in view of the increase in the cementitious value, which results from finer grinding. From the point of view of the manufacturers, however, the change has in some instances meant an increase in their manufacturing cost, owing to the higher grinding charges incurred in producing a finer cement. It should not be forgotten, however, that English manufacturers were pioneers in the matter of the finer grinding of Portland cement, and a large proportion of the cement manufactured in this country for years past has been so finely ground that it leaves a residue of only 6 or 8 per cent. on the 180 mesh sieve. Cement of this fineness, therefore, much more than complies even with the altered requirement.

The permissible residue on the coarser sieve (76 by 76) has also been reduced (from 3 to 1 per cent.), but this change does not possess the importance of the change in regard to the 180 mesh.

Tensile Strength, Neat and Sand.—Important changes have been made in these clauses: (1) in regard to the methods of gauging, (2) the minimum average breaking stresses at seven days have been increased (from 400 to 450 lb. per sq. in. for neat briquettes, and from 150 to 200 lb. for 3:1 sand briquettes), and (3) the tables giving percentages of increase required at twenty-eight days are eliminated and formulæ inserted for calculating the twenty-eight day requirement on the basis of the actual strength obtained at seven days. All these changes have an important bearing one upon the other.

In the former specification only "Mechanical Ramming" was forbidden, but now "No ramming or hammering in any form will be permitted," and "In filling the moulds with neat cement the operator's hands and the blade of the ordinary gauging trowel shall alone be used."

Much controversy has taken place as to the proper procedure for gauging test briquettes of cement. It is frequently overlooked that the object of the tests is to ascertain the comparative value of different cements, and that uniform methods are necessary for this purpose. One sometimes meets with the view that test briquettes should be made in the same way that "mortar is made by a mason." This, of course, is mere confusion of thought and can only lead to unreliable and erratic results. It has been very generally conceded that when making tensile tests of Portland cement, irregular and erroneous results can only be avoided by adopting such a method of filling the mould that the small breaking area of 1 sq. in. shall consist entirely of set cement and shall not be weakened by the presence of air or water bubbles. It is obviously impossible to compact into an area of 1 sq. in. more cement than is required to fill it, and a method of gauging which would ensure the breaking area consisting solidly of cement would be the one which would show the true strength, and would obviate fluctuations arising from reductions of the breaking area brought about by varying proportions of voids, due to the presence of air or excess water. This theoretically perfect briquette is obviously not obtainable in practice, but the method of gauging which approaches most nearly to it will give the most reliable and uniform results. Now, the method of gauging which is permissible under the wording of the revised specification will not result in so close an approach to the perfect briquette as was possible under the old specification, and it therefore follows that, generally speaking, lower results are to be expected from briquettes gauged as laid down in the revised specification. This expectation has been borne out in practice, and the breaking strains of a very large number of briquettes made by different expert gaugers give figures lower by anything up to 10 per cent. than briquettes gauged from the same sample, and by the same men, in accordance with the earlier specification. With this fact in mind it becomes apparent that the increase of the minimum seven day strain from 400 to 450 lb. is of more importance than might at first appear. It can, however, be said that, generally speaking, English Portland cements will show a margin over 450 lb. at seven days, even when gauged by the new method, providing the gauging is done by a qualified gauger.

The question of the increase required at twenty-eight days is a most important one, and it is possible that in some cases a cement of the highest quality, which gives an excellent seven day test, may fail to reach the twenty-eight day figure now required. For example, a cement tested under the 1910 specification, which gave 600 lb. per sq. in. at seven days, would have passed provided it showed not less than 5 per cent. increase at twenty-eight days, i.e., 630 lb. Under the 1915 specification, however, a cement giving 600 lb. at seven days would be required to show 667 lb. at twenty-eight days, or an increase of 67 lb.

Dealing now with the sand briquettes, there is not the same difficulty in securing a compact briquette by the new method prescribed as there is with the neat briquettes, although again a slight reduction is found in the strains obtained from one and the same cement as compared to the figures given by

the earlier method of gauging. It should be noted that here also the stipulations as to gauging which result in reduced strains are accompanied by the raising of the minimum seven day figure by no less than $33\frac{1}{3}$ per cent., i.e., from 150 to 200 lb. per sq. in. As is the case with the neat tests, the new formula for calculating the twenty-eight day strains is more severe than formerly, although not to the same extent. By-the-by, the dimensions given for the "Standard Spatula" are extremely curious—for instance, the length of the blade is to be 2.88 in. or 73.1 mm. It would be interesting to know how such a dimension was arrived at, as there appears to be no object in prescribing decimals of inches and millimetres for such tools, and it can hardly be believed that they can be commercially made to such microscopic accuracy.

In the case of both neat and sand tests something might have been done to eliminate irregularities due to gauging by prescribing that the results of the two briquettes giving the lowest strains should be eliminated in arriving at the average.

Setting Time.—The maximum final setting time for "Medium Cement" has been increased from two to three hours; the minimum final for "Slow Cement" being altered to agree. This alteration in the medium cement would appear to be a step in the right direction, although it is still a matter of some divergence of opinion as to whether three grades of setting time are really necessary in practice, and whether cements might not be better grouped under two heads, i.e., "quick setting" and "slow setting."

The prescription of a special point for determining the period at which the final set is to be read is certainly an improvement, and does something to eliminate the variations hitherto found owing to differences of opinion between one gauger and another.

Specific Gravity.—The type of flask to be used for making this determination has not hitherto been recommended, but a convenient type of flask is now illustrated, although not definitely prescribed as "Standard."

CEMENT DRAIN TILES IN ALKALI SOILS.

THE problem dealt with by the United States Bureau of Standards in a recent paper, an abstract of which we publish, is one which does not confront engineers in this country in the same form. The highly alkaline soils of the arid regions of the Western States have no parallel here. The principal conclusion derived from the year's experiments in the field is, however, applicable to other circumstances. The most important requirement in a mortar or concrete to resist alkaline salts is shown to be density or compactness. The greater the proportion of voids, the more easily salts are able to enter and the more injury they are able to bring about mechanically by crystallisation, which results in thrusting the particles of the mortar apart, and chemically by reacting with the lime of the cement. This is true, not only of alkaline waters, but of sea-water, many peaty waters, sewage, and waste industrial liquors. A dense, well mixed and rammed concrete, with a minimum of voids, has little to fear from the attack of salts; but a poor mixture, irregular and porous, soon disintegrates, and may become a loose, pasty mass under such conditions.

GOVERNMENT CERTIFICATION OF CEMENT INTENDED FOR EXPORT.

A LABORATORY for testing cement has now been added to the establishment of the Scientific and Technical Research Department of the Imperial Institute.

In certain countries, such as the Argentine, the Government has adopted an official specification for cement to which all cement intended for use in the construction of public works must conform. The Argentine Government also requires the certification of such cement by a laboratory recognised for that purpose by the Government of the exporting country.

H.M. Government has now recognised for this purpose the cement-testing laboratory of the Imperial Institute, at which analyses and tests will in future be conducted for British-made cement for contractors, engineers, manufacturers and others desiring to export cement to the Argentine and other countries in which a Government certificate is required. Firms requiring such assistance are requested to communicate with the Director of the Imperial Institute, South Kensington, London, S.W., who will supply information as to the conditions under which certificates are issued.

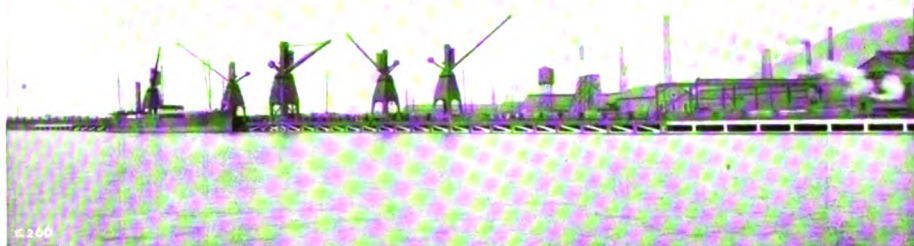


Fig. 1. General View of Wharf Front.

NEW REINFORCED CONCRETE WHARF AND JETTY FOR THE PORT TALBOT RAILWAY AND DOCKS CO.

NEW REINFORCED CONCRETE WHARF AND JETTY FOR THE PORT TALBOT RAILWAY AND DOCKS CO.

The following article describes some of the recent developments in one of our large shipping and industrial centres.—ED.

PORT TALBOT, which is one of the growing shipping and industrial centres of South Wales, has recently brought about very considerable improvements for the accommodation of the increasing amount of shipping which is directly connected with the industrial development of the district.

The wharf which we are about to describe, and which is constructed entirely on the Coignet system of reinforced concrete, is primarily intended for dealing with traffic from the adjacent large works of the Port Talbot Steel Co., Ltd. This wharf is situated alongside the dock main lines, and is provided with five very extensive through sidings for the manipulation of the traffic. It has a total length of 890 ft., which is sufficient to accommodate several large steamers. The whole of the work was carried out by Messrs. Watt Bros., contractors, of Cardiff, under the supervision of Mr. W. Cleaver, M.Inst.C.E., chief engineer to the company, and in accordance with his general plans and requirements.

The work was placed in competition amongst all the principal firms specialising in the design and construction of the various systems of reinforced concrete, and after very careful consideration of the various designs submitted the company decided to entrust Messrs. Edmond Coignet, Ltd., of Westminster, with the preparation of all the necessary working drawings of the reinforcement, etc., for the execution of this important work, which was started in the winter of 1912, both the wharf and the jetty being finished during the winter of 1914.

As shown in the accompanying plans and photographs, the wharf has been designed with a main frontage of 600 ft. in length and with two approaches of 145 ft. in length each. The width across the section of the main body is 45 ft. The width of the approaches measures from 16 ft. to

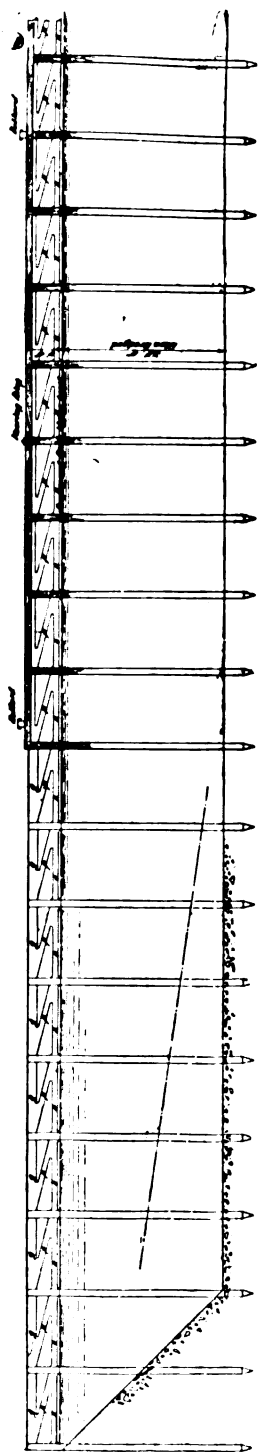


Fig. 2. Part Front Elevation.

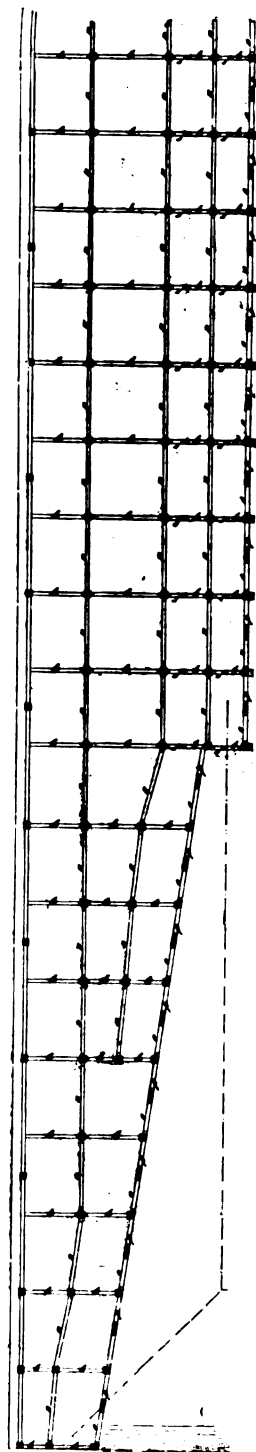


Fig. 3. Part Sectional Plan A-A.
NEW REINFORCED CONCRETE WHARF AND JETTY FOR THE PORT TALBOT RAILWAY AND DOCKS CO.

38 ft. The deck or platform is calculated to support three lines of railway to take 75-ton locomotives and several 2-ton travelling, hydraulically-operated cranes, each with a jib radius of 45 ft. and a total lift of 65 ft. In the central portion of the main body of the wharf a 15-ton hydraulic fixed crane has been erected, with a jib radius of 45 ft. and a lift of 60 ft. The wharf has also been wired and plugs fitted at convenient distances, for attachment to suitable leads on each crane, for the operation of electro lifting magnets when any material has to be loaded or discharged of a suitable character for their use, as both hydraulic and electric power is available from the company's own power stations.

The actual slabs of the deck, however, are only calculated for a super-load of 2 cwt. per sq. ft.

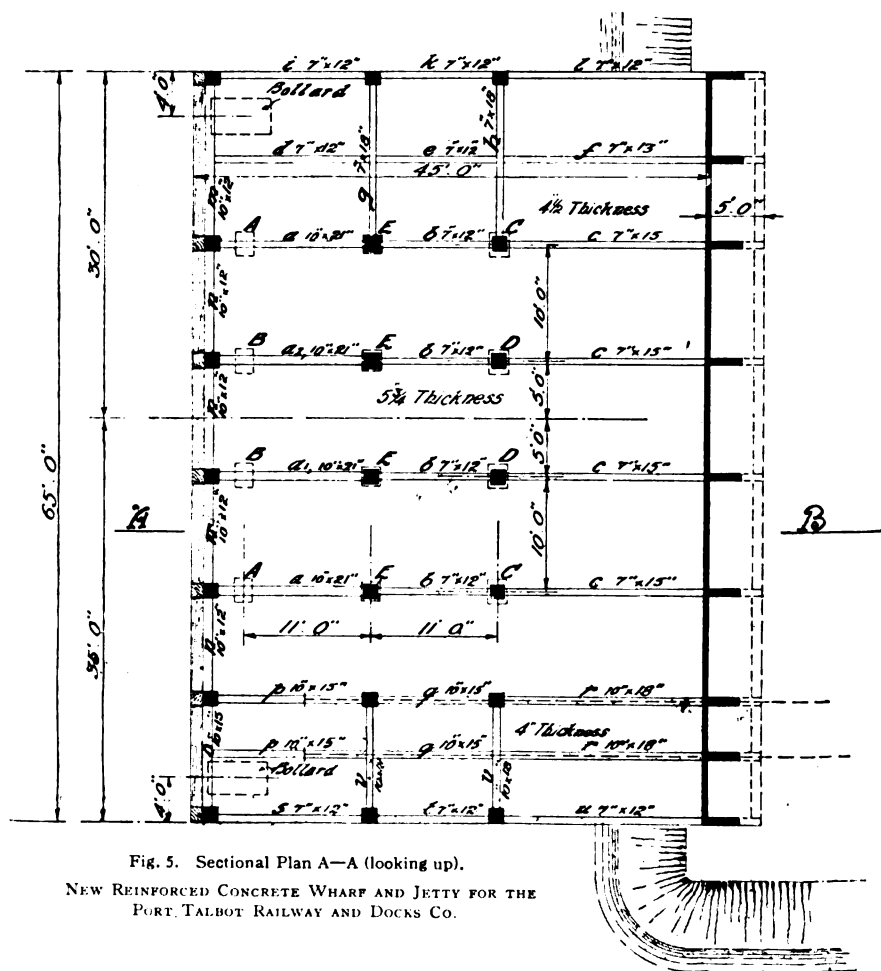


Fig. 4. General View of Jetty, showing Coal-Tipping Appliances.

NEW REINFORCED CONCRETE WHARF AND JETTY FOR THE PORT TALBOT RAILWAY AND DOCKS CO.

The total length of the wharf is divided into bays of 15 ft. centres, each bay being supported by a row of five piles. Before commencing the pile-driving operations the whole of the front was dredged to a depth of 32 ft. below the permanent dock water level. A certain number of timber piles were driven in order to find out exactly of what required length it would be necessary to make the reinforced concrete piles, all of which were constructed with a circular section 16 in. in diameter, and in accordance with the Coignet System. It was eventually found that the outer or front row of piles could not be driven more than 10 ft. 6 in. into the ground on account of the hard and compact nature of the boulder gravel bed into which these piles had to be driven. In order to leave 2 ft. at least of the piles above the water

line it was necessary to make the first and second rows of piles 44 ft. 6 in. in length, the third row 42 ft., the fourth row 30 ft., and the fifth row 23 ft. The necessary set was obtained by means of a 2-ton monkey falling from a height of 4 ft. The whole of the piles were driven by means of a steam monkey, the pile driver and boiler being supported on a movable platform resting at one end on a barge and at the other on the shore.



All the reinforced concrete piles were strongly braced together a few inches above the water level, this precaution being essential on account of the considerable height of the front row of piles. In order to prevent the latter being damaged by the continual impact of ships from 8,000 tons to 10,000 tons dead weight capacity, a pitch pine fender pile, 14 in. by 14 in., was driven down into the gravel in front of each of the reinforced concrete piles, and horizontal rubbing pieces or caps were provided throughout the

whole length of the front. Regarding these fenders, the company's engineer also designed that the timber fenders should only be in contact with the wharf at two points—i.e., at deck level and at water level—or opposite the lower framing, leaving a space elsewhere between the concrete and the timber pile of not less than 2 in. to 3 in. This formed a very effective cushion, ensuring that with any ordinary impact the natural elasticity of the timber would come into operation, whereas with any extraordinary blow the timber pile would deflect the amount of 2 in. or 3 in. space and touch the concrete pile before any fracture could take place. The concrete piles are therefore not subject to impact, except very occasionally.

NOTE.—The timber fendering is secured to the wharf only at deck level, where everything is easily accessible. No fastenings were permitted under water.

The two front rows in particular have been very strongly braced together by means of diagonal struts, and the entire work, which has now been in use for about a year, presents a very strong and monolithic appearance. All the piles were made on the site adjoining the works, and lifted into position by means of a 5-ton steam travelling crane. All the wharf cranes being operated by hydraulic power,

it was necessary to arrange suitable ducts or chases in the deck in order to accommodate the pipes, etc. Very strong bollards and rings have been placed alternately at a distance of every 60 ft. A number of hydraulic capstans have also been provided for at a distance of every 160 ft. throughout the length of the front, and, in fact, this wharf has been very carefully studied to facilitate the traffic and to expedite the loading and unloading of ships and wagons in the most up-to-date manner.

The approaches are fitted with wrought iron handrails, ladders, etc., and special recesses have also been left in the deck for the (turnover) hydraulic capstans. The track of the travelling cranes, and the railway running underneath the latter, are laid on timber way beams fixed directly on the reinforced concrete deck. The two back railway lines, however, consist of ordinary cross-sleepered roads, laid on ordinary ballast, the wharf decking constituting the formation. The railway lines and the construction work of the wharf are calculated to take heavy locomotives weighing over 75 tons.



Fig. 6. Making Reinforcement for Concrete Piles.

NEW REINFORCED CONCRETE WHARF AND JETTY FOR THE PORT TALBOT
RAILWAY AND DOCKS CO.

The whole of the reinforced concrete work in this wharf was carried out with a mixture of concrete of approximately 1, 1½, 3, the aggregate consisting of clean Nantes sand and granite chippings. All the bars, which have been bent and arranged in accordance with the system adopted, are of plain circular section and of mild steel. The reinforcement of the piles is composed of a certain number of vertical bars surrounded by rings, or hoops, of small section. All the horizontal and inclined braces are formed by means of four longitudinal bars, varying in diameter according to their position in the work, and bound together by means of ties, or stirrups, of small section.

Two different types of reinforcement were used for the beams. Some were made by means of units containing a top and bottom bar connected together

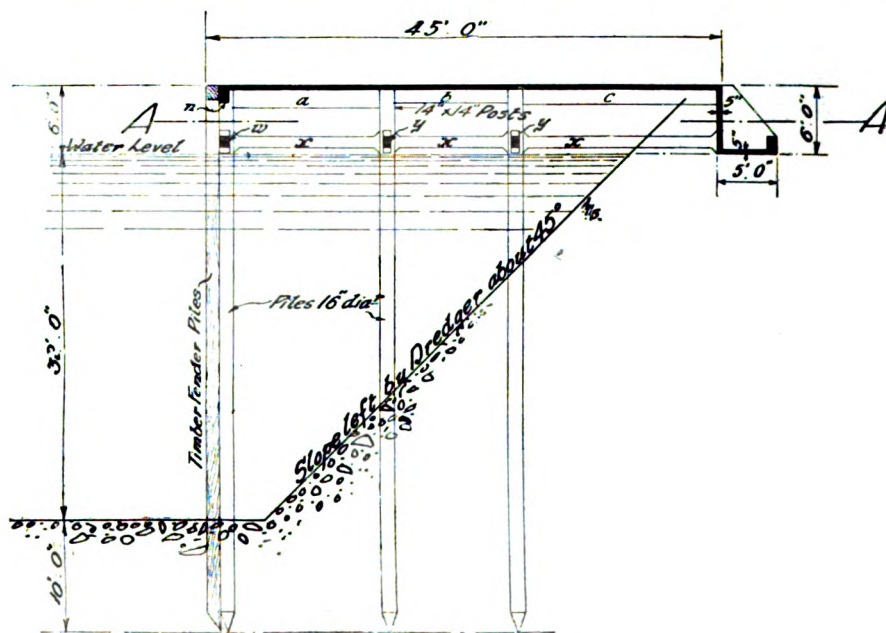


Fig. 7. Cross Section.

NEW REINFORCED CONCRETE WHARF AND JETTY FOR THE PORT TALBOT RAILWAY AND DOCKS CO.

by stirrups placed gradually closer together nearer the points of support, in accordance with the shearing stresses. This method was adopted throughout the whole length of the longitudinal beams supporting the railway lines and crane tracks. The other method, which was adopted for the transverse beams, consists of an arrangement of groups of seven bars having their ends bent up at an angle of 45 deg., to resist the shearing stresses. These shear members are hooked to a top longitudinal bar in such a manner that each of the bars is mechanically fixed at both ends to resist the diagonal tension. Additional vertical stirrups, however, are provided to each of these units, or groups of bars, in order to obtain the necessary resistance to shearing stresses.

All the various units, either composed by means of stirrups and straight bars, or by means of bent bars, were prepared beforehand and tied together by

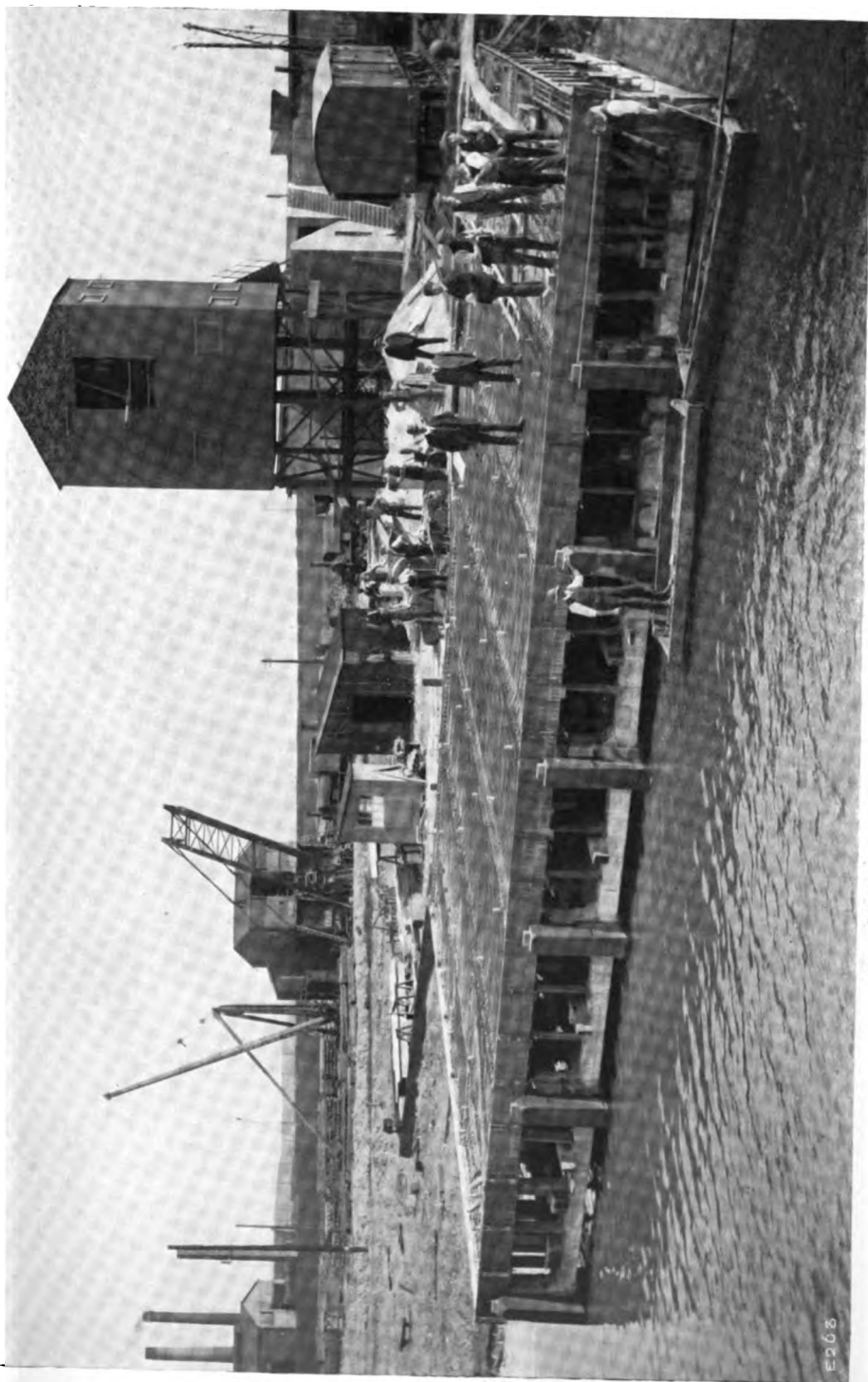


FIG. 8. Reinforced Concrete Jetty at Port Talbot during Construction.
NEW REINFORCED CONCRETE WHARF AND JETTY FOR THE PORT TALBOT RAILWAY AND DOCKS CO.

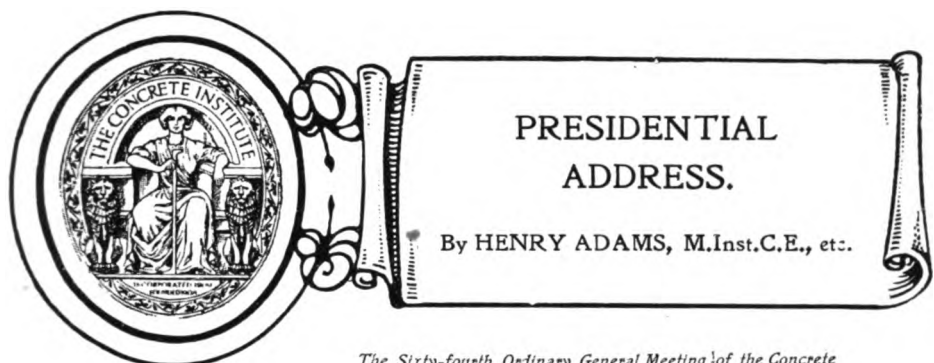
means of annealed wire to form complete units, this being one of the special features of the reinforced concrete system here adopted.

The reinforcement of the deck is simply composed of a meshwork of principal and secondary bars arranged in a suitable manner to take up the tension wherever it occurs.

The jetty, which was constructed concurrently with the wharf, is for the purpose of supporting a modern coal belt tipping plant. The total length of the jetty supporting the coal belt and tipping appliance is 65 ft., with a width of 45 ft. The front row of piles was driven to a depth of 8 ft. into the gravel bottom, so that they had to be made 42 ft. in order to allow of a sufficient length above the level of the water. The second row of piles was made 37 ft. 6 in. in length, and the third row 35 ft. In the case of the jetty, the back of the wharf rested on good solid ground, and therefore a substantial counter-fortified retaining wall, as shown, was adopted in lieu of piles. These piles, 16 in. in diameter, were made in the same manner as those of the wharf. In fact, the general arrangement of the reinforcement is very similar to the method adopted for the wharf, with the exception that it has not been necessary, in this case, to introduce any diagonal bracing. Timber fenders have also been provided, a duplicate method of that adopted in the wharf, to prevent injury to the reinforced concrete work, and a suitable number of bollards and rings, and other necessary fixtures have been devised, to facilitate the rapid loading of the coal, in accordance with the most up-to-date and modern methods. A screenings and general stores siding has also been provided for on this jetty, as shown.

The whole of the permanent way for sidings and crane road, all the hydraulic piping, valves, etc., the electric wiring and magnets, and the hydraulic capstans, bollards, and rings were provided and laid in by the Port Talbot Railway and Dock Company's own men after the reinforced concrete work had been completed.

The hydraulic capstans are of the well-known turnover pattern supplied by Sir W. G. Armstrong Whitworth and Co., Ltd., and the hydraulic cranes were constructed by the East-Ferry Road Engineering Co., Ltd., Millwall, London.



The Sixty-fourth Ordinary General Meeting of the Concrete Institute was held last month at Denton House, Westminster, when the President, Professor Henry Adams, M.Inst.C.E., delivered his second Presidential Address. We give below an abstract from the Address.—ED.

INTRODUCTION.

AFTER some general remarks relative to the membership of the Institute and the meetings held in the past year, the President went on to say:—

L.C.C. Regulations for Reinforced Concrete.—The labours of the Council of the Concrete Institute in the matter of the Reinforced Concrete Regulations, extending over a period of six years, have at last borne full fruition. In the draft of the Regulations which was submitted to the Council for final approval before they became law, the whole of the Council's recommendations were found to have been adopted, with very slight modifications here and there. Very prolonged attention and a considerable amount of thought and anxiety have been devoted to the subject, and the result is one of which the Institute may justly feel proud. Reinforced concrete construction carried out in accordance with these Regulations will be absolutely safe, and, so far as can be judged at present, its effective life will only be measurable by centuries.

The completion of these Regulations has been, for us, the great event of the year, and our heartiest thanks are due to the London County Council. No previous Building Regulations have been compiled with so much care or with so honest an endeavour to meet the reasonable views of all parties. The exceptional position which the Institute holds in regard to the subject secured a certain amount of preferential consideration for their views, and our good friend, Mr. E. Fiander Etchells, was a sympathetic medium.

Other Work.—We have no great work in hand now, but we cannot afford to stand still. If we wish to progress we must look around and see where improvement can be made.

The arrangement of our present committees does not seem to me quite the best that could be desired. For example, the Science Committee should, in my opinion, be more comprehensive. It should undertake the whole business relating to the principles of, and constructional practice in, reinforced concrete, with power to appoint sub-committees, such as Cement Tests Sub-Committee, Research Sub-Committee, Failures Sub-Committee, besides others from time to time dealing with specific subjects such as notation, specifications, quantities, pile-driving, foundations, etc.

Many interesting papers have been read and discussed at our evening meetings, and I need not recapitulate them. We are glad to welcome papers in all branches of theory and practice in structural engineering, particularly those relating to practice, and when the author has the courage to state his opinion freely the discussion is always vigorous. There must of necessity be differences of opinion, and it is only by free discussion that we can arrive at the truth. Our motto should be "Try all things, hold fast that which is good."

Steel.—In view of the present predominance of steel as a material of construction, and the exceedingly satisfactory way in which it behaves in general, it is somewhat curious to look back and note the distrust with which it was regarded by

engineers of the last generation. When Sir Henry Bessemer suggested the use of steel for rails, a prominent engineer remarked that those who wished might use steel, but that personally he was not desirous of being indicted for manslaughter. The good qualities of the material were, however, gradually recognised, but at the same time there were periodically expressions of alarm at certain mysterious failures, of which at the present time we hear little or nothing.

Mr. Osmond has pointed out that the quality of structural steel is really a function of three variables—viz., its chemical composition, its structure, and its freedom from initial stresses. All of these variables may differ materially, even in bars rolled from a single ingot. Experiment shows that while the ingot is solidifying, the carbon, sulphur, and phosphorus tend to concentrate in the centre of the upper third of the ingot. Indeed, a content of phosphorus, on the average but 0.06 for the whole ingot, may, through segregation, be five times this figure in parts. The structure of a steel depends mainly on its treatment from the ingot up to the finished bar. Much light has been thrown on this matter by the microscopic examination of the polished and etched surfaces of the metal, whilst the knowledge of the properties of solutions worked out in other departments of science have permitted of some explanation being given of the peculiarities of structure found. Mere heat treatment is alone sufficient to alter considerably the structure of a bar.

We have not yet settled the relative position of the architect and engineer in steel-framed and reinforced concrete buildings. The author then went on to quote from papers read by Mr. Cyril Cocking and Mr. Brown on the relation between the architect and the engineer. An extract is then given from an editorial in these pages on the same subject (see issue, April, 1915), in which emphasis is laid on the necessity of closer collaboration between the two.

The author then went on to say:—

I have previously expressed the opinion that there should be more collaboration between the architect and the engineer, and our joint meetings will go far to remove any difficulties that may arise in practice. It is said in some quarters that there is too great a tendency for architects to look upon concrete specialists and constructional designers as merely better-class craftsmen, but as craftsmen to be entirely subordinate to the architect. I can only say that architects are among my best friends, and I cannot complain that they have shown any lack of consideration in any views I have put before them.

REINFORCED CONCRETE.

Reinforced Concrete Construction Work carried out during the year.—

Although this has been one of the most eventful years in the history of the world, there is not very much to chronicle in the domain of reinforced concrete construction. Of the larger buildings in this material that have been completed during the year, there is the very large and effective warehouse built for Messrs. Ralli Brothers, at Stanley Street, Manchester. Another large warehouse is that at Laisterdyke, Bradford. A two-million gallon reservoir at Runcorn and many smaller works and buildings have also been built on the same system during the year.

Owing to the moist atmosphere to be resisted, reinforced concrete is exceptionally advantageous for swimming baths; good examples will be found at Croydon and Hammersmith. For water towers it has also many advantages, although perhaps the tank portion could be constructed more cheaply in steel alone. A water tower at York, said to be the largest in Great Britain, was constructed to hold 300,000 gallons. Many other interesting works have been carried out, but if you are subscribers to that useful monthly magazine *Concrete and Constructional Engineering*, you will be kept informed of what goes on.

A most unusual piece of engineering construction is nearing completion at Baltimore. The work consists essentially of converting an old brick-lined water tunnel, in which the flow was by gravity, into a pressure conduit operating under a head of 35 lb. per square inch. The novelty lies in the use of huge separately moulded reinforced concrete pipes, the largest having a diameter of 9 ft., to form the new tunnel lining. As the existing tunnel is only 12 ft. in diameter, it is clear that the work had to be carried on in cramped quarters, and, in addition to the difficulties inherent in placing 10½-ton sections of pipe with so little working

room, there is the obligation of shutting down operations every few days and using the conduit to deliver water to the city. The valves are then closed and the tunnel drained in order to allow the work of lining to proceed. One of the features of the construction is a specially designed car, which carries the ponderous pipes into the tunnel, and by means of adjusting equipment sets them accurately to line and grade. The concrete pipes are of particular interest, for they are claimed to rank among the largest precast units ever built. It is also unusual to use reinforced concrete to resist such a great internal pressure as is found in the Baltimore tunnel conduit.

Reinforced Concrete for War Purposes.—The *Engineering Record* of New York, in a recent leading article, states that every war affords great scope for the skill of the engineer, and the present terrific struggle has already presented some questions of startling significance, reference being made to the gigantic tasks of transportation which are being performed. Not less in importance, however, are some other problems which demand immediate attention for efficiency and solution. One of these is to find the material of maximum effective resistance to high explosives. It seems to have been demonstrated effectively that concrete, which has been the main standby in modern fortifications, is almost useless in its customary form. The turrets of permanent concrete forts were rendered useless by a few rounds of shells charged with high explosives. From the photographs of the demolished works it appears that, even when the turrets themselves were not demolished, they were fairly blown off their foundation by the complete shattering of the concrete bases. On the other hand, temporary earthworks, and one or two forts heavily supported with earth, appear to have resisted attack by high explosives measurably well. A shell penetrating a few feet into concrete rends the whole mass, while in earth it merely opens a crater, which is partly filled in again by the falling dirt. The questions now to the front are whether any reinforcement of the concrete will be of service, or whether a proper choice of earth construction will so muffle the explosives as to make them comparatively ineffective. A few feet of loose dirt, or even of snow, will stop a bullet which would pierce a half-inch iron plate, and the same principle seems to hold good for heavy projectiles. Lieut.-Col. Roustam Bek, the talented Russian expert, whose interesting war articles in the *Daily Express* may be known to many of you, says: "Engineers do not know any kind of material stronger than concrete and steel for cupolas and other bulwark constructions, but those, as far as we can see, are not sufficiently solid to withstand the terrible fire of the monstrous 16-inch guns. It has always been so, and it is at present, that the technical progress of the artillery has been more rapid than the progress of the engineering art, and the rôle of the fortress will be unsatisfactory unless its fortification be made impervious to the destructive action of siege artillery."

It is possible, however, that it may be found useful for the interior of forts with sufficient earth protection, as it has already proved valuable in roofing trenches and dug-outs where the earth can be placed on top, and also for side walls in fighting trenches when sufficient time is available.

In this country we have found reinforced concrete useful for huts for training camps, and a large number of them have been constructed with expanded metal.

With further reference to the effect of reinforced concrete in military operations the following is *à propos*. A writer appointed by the Home Office to observe and describe the effect of the Zeppelin air raid over London on October 13th, in the course of his description, said:—

"The third area contains two damaged business premises, the first of them a large and modern building constructed of reinforced concrete, and with a steel and concrete roof and flooring. Two bombs dropped in this building, one of them actually on the roof, and one on the pavement immediately beneath the doorway.

"The bomb on the pavement appears to have exploded sideways; at any rate, the damage done, which consisted chiefly of broken glass and plaster, occurred mainly in the houses on the other side of the street. The bomb which dropped on the roof of the building itself did little damage."

This information is very satisfactory, in view of the very considerable damage done when an explosive bomb dropped on a brick building.

In the reconstruction of the numberless bridges on the Continent destroyed by

the war, reinforced concrete will no doubt be largely used. It is a fine opportunity, and I hope some of our members will, later on, be able to give us a good paper on the subject. A reinforced concrete bridge at Soissons was blown up by the Germans in their retreat from that town, but owing to the great strength and toughness of the material it defied all attempts at complete destruction, such as was readily effected in the case of the other bridge at Soissons, which was of metal. The former still crosses the river after a fashion, and gangways laid over the ruins give access to both sides.

Another instance of the resistance of reinforced concrete to explosives was given in June last, when an attempt was made to blow up a building in Ontario, where military clothing was being made for the British Army. From information gained by the authorities the charge consisted of twenty-six sticks of dynamite, but the only result was to blow out ten to fifteen feet of a brick retaining wall, damage a concrete slab forming a footway, and shatter the glass of fifty to seventy-five window sashes. The owner expressed the opinion that if the building had been in any other material than reinforced concrete, it would have been a total wreck.

Fire Resistance and Reinforced Concrete.—In my previous address I called attention to the fireproof qualities of reinforced concrete construction, and during the past year these qualities have been exemplified on more than one occasion.

A testimony as to the stability of reinforced concrete buildings under the action of fire comes from the accounts of the fire which destroyed a large portion of the plant of Mr. Thomas A. Edison, at West Orange, New Jersey, on December 9th last. The first account erroneously stated that the concrete buildings, supposedly fireproof, had been destroyed. These accounts were, according to the *Architects' and Builders' Journal*, very inaccurate. The seven concrete buildings attacked still stand, while about them were the tangled and hopeless ruins of one frame and six brick buildings. The fire was a very fierce one, and five chief reasons are assigned for the extent of the disaster and the rapid progress of the fire. These are: (1) The highly inflammable character of the contents of the buildings; (2) the inadequacy of the water supply; (3) the fact that the window openings were fitted with wooden sashes and plain glass; (4) lack of fire walls; (5) lack of automatic sprinklers. With the exception of the upper floors of certain sections the concrete buildings remained in good condition and the salvage was proportionately extensive. Columns spalled here and there, without, however, exposing the reinforcing rods, and occasionally, but not often, the beams also spalled. In general, the more destructive effect of the fire upon the columns than on the beams was noticeable. The effect of the fire on the columns extended generally to a depth of about 1½ in. Except where the fire was very severe, the concrete did not come off and the rods were not exposed. Longitudinal cracks were found on many of the columns, the reinforcing of which consisted only of four vertical rods without horizontal ties or hooping. A considerable proportion of the damage to the columns is attributed to the streams of water which were directed upon them when heated, and it is the belief of the officials that very little damage would have been done had the water not been directed upon them after they had become very hot. Three specialists employed in the investigation have reported that 87½ per cent. of the concrete construction remained in first-class condition and the work of restoration was promptly put in hand. No better recommendation of reinforced concrete could be given than the evidence afforded by this fire.

Stability against Earthquakes.—In my former presidential address I also referred to the stability of reinforced concrete against earthquakes. In the earthquake at Avezzano, in Italy, on January 13th, 1915, out of 12,000 inhabitants only a few hundreds escaped immediate death. Avezzano was situated on the slope of a supposedly extinct volcanic crater, which for centuries had been filled with water, and was known as Lake Fucino. In the middle of the last century a rich papal banker, Prince Torlonia, went to enormous expense draining the lake. An area covering forty square miles became a fertile agricultural land, devoted to raising sugar beet, and dotted with scores of prosperous villages and towns, of which Avezzano was the chief. To-day only one building remains intact. This is a house built of reinforced concrete on the road from the station.

Cement Industry.—Of course we are all interested in the serious development

of the cement industry, and the progress of building in concrete and reinforced concrete.

Owing to the shortage of German and Belgian supplies of Portland cement, a good deal of attention is being given to the manufacture of this material abroad, in places where limestone, shale, or clay and coal can be obtained of suitable quality and in sufficient quantities to make the manufacture likely to be successful and remunerative.

Why is it (says the *Engineering Record*) that in two concrete buildings apparently constructed under identical conditions, built by contractors of equal intelligence and integrity, from concrete composed of similar aggregates and the same brand of Portland cement, the floors in one will turn out hard, firm, and resistant to abrasion, while in the other ordinary usage will result in dusting? The fact that numerous dustless concrete floors have been laid seems to indicate that the trouble must lie in the selection, proportioning, mixing, placing, or finishing of the material. The procedure and proportions described below have given excellent results. For first-class work a rich mixture is desirable, say one in which the aggregate consists of granite or other hard stone, screenings graded from $\frac{1}{4}$ in. down to the finest, and crushed stone of equal quality passing a $\frac{1}{4}$ -in. ring and retained on a screen having a $\frac{1}{4}$ in. mesh. All trowelling and finishing of the floor surface should be completed within two and a half hours from the time the materials leave the mixer. This necessitates mixing the material to such consistency that the mortar has to be scraped from the wheelbarrows, and will hardly flatten out when dumped upon the floor wet enough so that it can be "struck off" with little difficulty when spread out with shovels. The floor usually is in a condition to be trowelled for the last time within an hour and a half or two hours after the finishing course has been mixed. When sufficiently hardened to prevent pitting, the floor should be sprinkled with water until 2 in. of sawdust can be thrown on the surface without injury. The sawdust should be thoroughly wetted and kept moist by sprinkling for a period of two weeks.

A mixture of 15 or 20 lb. iron dust with 100 lb. Portland cement added to twice that quantity of sand applied as a top coat 1 in. thick to a concrete floor is said to render it hard and durable.

Concrete Mixtures.—What may be called the standard mixture for concrete in reinforced work is 1 part Portland cement, 2 parts sand, and 4 parts larger aggregate, by volume, commonly known as a 1 : 2 : 4 mixture. There is a very objectionable practice on the part of some people who ought to know better, to call this a 1 to 6 mixture, with the result that a builder may have some ground for assuming that he complies with the requirements by using 1 part of cement to 6 parts of combined sand and larger aggregate, which would approximately be a 1 : 3 : 6 mixture, but there is no guarantee of the proportion unless the materials are mixed separately, 90 lb. of cement being taken as equal to 1 cu. ft.

The study of photo-micrographs of concrete shows that, although standard proportions of the materials are used, the resulting mixture will not be perfect unless these proportions are suited to the voids in the aggregate, and it thus becomes essential to ascertain the voids in any proposed aggregate before proceeding farther. The quantity of water required in mixing concrete will depend somewhat upon the materials, but it appears from a series of experiments that the best proportion is 25 to 27½ per cent. of the mass, although this is rather wetter than we usually make it.

Failures in Concrete Structures.—In view of the remarks upon failures of concrete structures in my last address, the paper read by Mr. R. S. Greenman before the American Society for Testing Materials, and published in the last December number of *Concrete and Constructional Engineering*, is particularly interesting. He stated that the causes approximated to 90 per cent. due to poor workmanship, 8 per cent. to poor aggregates, and 2 per cent. to poor cement. Fortunately, the larger the work the greater the care that is taken, so that the failures chiefly lie with the smaller and less important constructions. He gave instances of failure due to impure water, sand, and stone, all of an exceptional character, and for that reason more liable to escape attention. He considered the failures due to poor workmanship to be so common as not to need detailed examination and summarised the chief appearances of improperly mixed concrete. In conclusion, he recommended both laboratory

tests and field inspection, so that the conclusions of the investigator might be founded upon a consideration of the failure from every point of view, whether the object of the examination was to place responsibility or to guard against future failures. From the engineer's point of view the latter is the sole point of interest.

In a case of defective workmanship 30,000 cu. ft. of water shot in a cataract from a tank on the roof of a mill at Bradford on February 12th, 1914, when one of the end walls collapsed and swept scores of bales of wool out of the building. The mill, known as Cashmere Works, was completed six months before. It was built of reinforced concrete, and was 150 ft. long, 70 ft. wide, and about 150 ft. in height, divided into six stories. Across the entire top of the building was a tank capable of holding 3 ft. of water for the fire-sprinkling apparatus. One end of the building burst outwards with a loud crash, the water-tank collapsed, and the water rushed through the building and out into the street, carrying with it many tons of wool and cashmere. The street slopes considerably, and the water poured down in a flood so deep that bales of wool floated along for two hundred yards. No workmen were in the part of the building where the accident happened, but only a little while before a large number had left. Practically the whole of one end of the building was torn out by the weight of wool and the force of the water.

CONCLUSION.

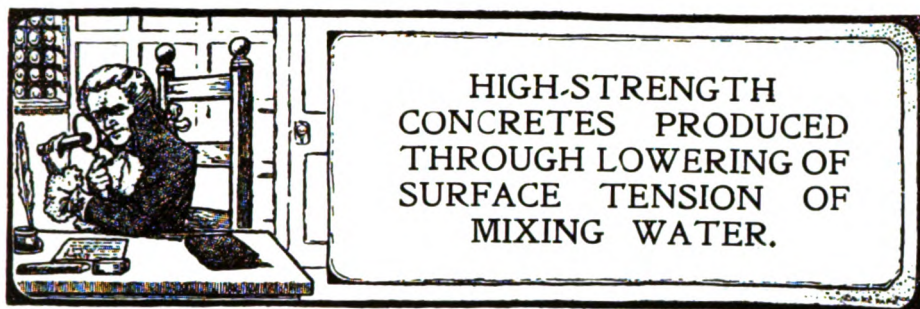
The time has gone by when engineers are justified in regarding their knowledge of how to do things as professional secrets. Looking at the matter strictly from the standpoint of individual selfish advantage, and not that of professional welfare, it is not desirable to have the same mistakes repeated. Engineers are not usually retained by other engineers, but by men without technical knowledge. These men, learning that certain kinds of work or certain classes of plants have not been successful, are naturally prejudiced against them, and look with suspicion on the advice of any engineer on such subjects. The best way to have the engineering profession strengthened is to prevent inferior or bad engineering work, and the best way to accomplish this is to tell freely what proves successful and what proves unsuccessful, so that everybody may profit, so far as in him lies, by the knowledge of all. No engineer will lose standing among his colleagues or be ranked any less competent by business men for frankly putting on record a statement of his non-successes. On the contrary, he will receive the approbation of all thinking men, for he will warn others away from a repetition of his unsuccessful work, which is a most desirable thing.

In Mr. R. S. Greenman's paper we read among the opening remarks: "Concrete is said to be its own best inspector, and it is a well-known fact that defects in concrete will sooner or later make their presence known." Now, I do not think much of an inspector who only makes defects known when it is too late to remedy them, but the important point is that defects will show themselves sooner or later. He goes on to say: "For every fault there must be a reason." That sentence ought to be taken as "the text" by our Investigation Committee, and no case should be allowed to rest until the "reason" or primary cause is found.

We must not be discouraged if we are unable to learn as much as we should wish. One of the greatest men who ever lived, Sir Isaac Newton, when congratulated by his friends upon the extent and greatness of his discoveries, replied: "I know not what the world will think of my labours, but to myself it seems that I have been but as a child playing on the shore, now finding some brighter pebble, now some more variegated shell, while the vast ocean of truth lay unexplored before me."

VOTE OF THANKS.

At the conclusion of the address a vote of thanks was proposed by Sir Henry Tanner, C.B., seconded by Mr. E. Fiander Etchells, M.Inst.C.E., and supported by Mr. Osborn C. Hills, F.R.I.B.A., District Surveyor for the Strand.



Showing How Microscope Has Led to Discovery of Reason for Low Percentage of Hydration of Cement, and Describing Experiments for Increasing the Percentage.

By NATHAN C. JOHNSON,

Consulting Engineer, New York City.

Conclusion. — In reprinting these articles we would state that the opinions expressed are those of the author, and the Journal as such does not necessarily associate itself with the conclusions arrived at. The articles are reprinted by the courtesy of the "Engineering Record," U.S.A., and the illustrations have been placed at our disposal by the author. —ED.

To be of the highest value, research in any line must find, directly or indirectly, an economic application. The improvement of concrete in quality, cheapness and strength is an economic goal of no mean order; and the measure by which all concrete investigation must be judged will be the degree of improvement that will ultimately be brought about in the commercial product. Any change from established ideas and procedure, no matter how desirable, is necessarily slow of acceptance, but once the feasibility of accomplishing desirable ends is proved, time alone should be the only factor influencing the general adoption of the new methods.

Previous papers of this series may have appeared iconoclastic. Concrete has been dealt with not as a universal, ideal material, but as one excellent in spite of its deficiencies and susceptible of the very greatest improvement, once these deficiencies are recognised. Believing that continued labouring under removable limitations is an inefficient policy, and finding further that many of these deficiencies of concrete are due to very definite causes, certain weaknesses and faults have been pointed out at some length to emphasise human fallibility and carelessness as the responsible factors, rather than the materials themselves. Especially is this true as regards processes such as are involved in mixing a finely divided substance with water. Apparently one of the simplest of operations, this is yet one of the most difficult to accomplish, as was demonstrated in Article 5 of this series. In the course of the present paper, its extreme importance to the strength of concrete, by reason of inefficient and insufficient wetting of the cement, will be particularly emphasised.

It will be remembered that certain lumpy formations found in commercial concretes have been previously designated as groups of particles of unhydrated cement. It has also been stated, with the prevalence of these groups as a partial basis, that not more than 20 per cent. of the total cement added to concrete is effectively used. Obviously, these statements require most rigid proof, but if they are proved, the future of concrete construction is bright with promise for wider usefulness by a simple development of the material already at hand. For instance, the present effective percentage of cement allows us to use our accustomed stresses, say, 600 lb. per square inch in compression, with the strength in tension considered as zero. If the compressive strength alone could be doubled, regardless of the possible tensile strength

developed, columns and beams could be stressed to *twice the present allowable limit*; and corollary to this, though perhaps not in the same ratio, the size of members and the quantity and, therefore, the cost of materials required for them could be decreased. Or to be more sanguine, if it were possible to impose *three times*, or *four times* the present allowable stress, the future possibilities of concrete as a structural material would almost stagger the imagination.

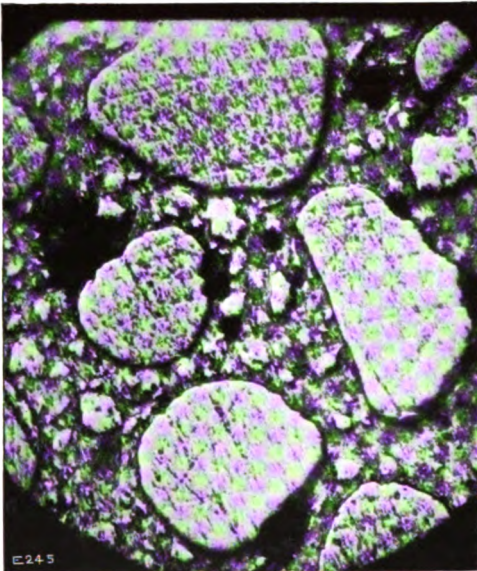


Fig. 29. Unhydrated Cement in 1:3 Standard Sand Briquette.
90 days old. 50 diameters.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

either hydrated or unhydrated. By direct vision through the microscope, the lumps between the sand grains can be further identified by their characteristic brown, or brownish-green, or greenish-black colour, as unhydrated cement; and by elimination, the white mass in which they lie embedded can be nothing else than partially or wholly unhydrated portions of the same material.

If a planimeter were used to integrate the areas of this section, as in the method of analysis before described (*Concrete and Constructional Engineering*, September issue, p. 450), the basis for the quantitative estimate of 80 per cent. of cement remaining unhydrated could be understood. The appearance of sections such as this led to the undertaking of this research on the improvement of hydration some four years ago; and since that time, the investigation has been pushed unremittingly, a portion of the results so obtained being presented in these pages.

As seen in this photograph, however, the lumps of cement do not appear similar to those honeycomb formations found in concretes which have been previously designated as groups of unhydrated cement particles. This is because in Fig. 29 the magnification is low and also because the structure of the cement lumps has not been developed in polishing; but if the latter is done and examination made at a higher power, the structure of (a) in Fig. 30 is revealed; while contrasted with this is the structure (b) of the same figure, found in a commercial concrete, taken from a dock

All these visions must fade, however, unless this low efficiency of use can not only be proved, but means for improvement demonstrated. This requires, as a first step, a determination of the percentage of total cement represented by these lumpy formations, and their identification as unhydrated cement.

IDENTIFICATION OF UNHYDRATED CEMENT.

To gain some information in this regard, it may be well to section and polish a mortar briquette made from known materials—for instance, standard Ottawa sand and an approved grade of cement. A photograph of a section of such a briquette, stored under water for 90 days after mixing, shows as in Fig. 29. In this photograph identification of materials is made easy, as only standard sand and cement and water entered into the briquette; and as the standard sand grains are unmistakable because of their form and their size, the material can be nothing other than cement,

wall in New York harbour. And as before, visual examination further confirms the identity of these materials whose structures are compared, by showing the charac-

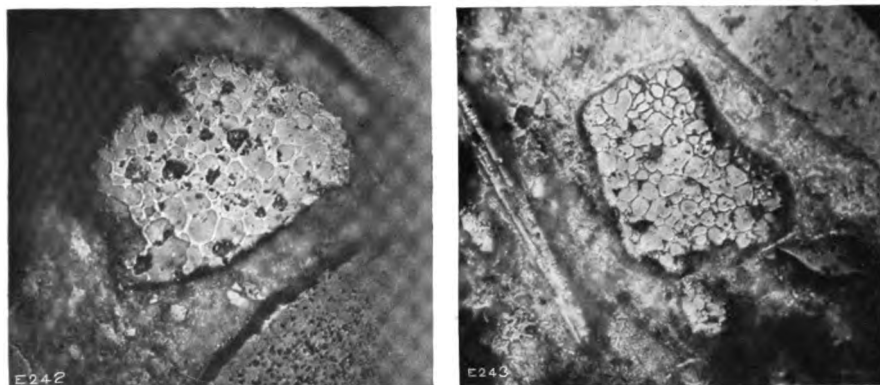


FIG. 30. (a) Ottawa Sand Mortar; (b) Dock Wall Concrete.
Groups of unhydrated cement particles: 300 diameters.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

teristic greens and browns and blacks of unhydrated cement, which are lost in the photographs.

FORMATION OF UNHYDRATED GROUPS.

Assuming, then, the identity of these formations as grouped particles of unhydrated cement to be established, it becomes of prime importance to inquire as to how and why such groups occur, and as to how their unhydrated condition is maintained unchanged, regardless of conditions of moisture, in order that means may be found



Fig. 31. Piece of Clinker in Old Concrete.

The crystals were formed by breathing on the polished surface.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

to prevent their future formation and to insure their component particles entering actively into the concrete.

Evidently, these particles were more or less wetted at one time, for the network boundaries are found by examination to be composed of a mixture of wholly or partially hydrated cement, but why the hydrating action stops short after the initial moistening is less evident, but most important to determine. Obviously cement and water are the two reacting substances; and to secure thorough reaction there must be thorough contact between them. Any grouping, therefore, that would prevent a proper access of water to the cement would result in an incomplete reaction, with consequent detriment to the quality of the concrete.

To understand the causes of formation of these groups in concrete it is necessary to consider briefly the general chemical nature of cement.

All Portland cements contain, in varying amounts and combined in devious ways, a number of chemical "radicals"—CaO, MgO, SiO₂, Al₂O₃, Fe₂O₃, SO₃, and so on. The presence of some of these compounds is due to the composition of the raw

materials, while that of others, notably a certain portion of the CaO and SO_3 , is attributable to the secondary addition of gypsum to the finished product.

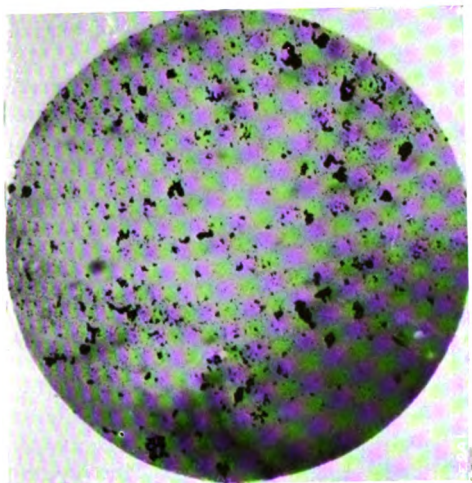
SETTING AND HARDENING.

It follows from the complex nature of cement that the reactions occasioned by its admixture with water are themselves complex. Indeed it is doubtful if these reactions are even now fully understood. Many are yet matters of controversy, but regardless of their exact chemical nature, it is well established that in the passage from the semi-fluid state of gauged cement to that of the hardened product, there are two distinct stages, the first known as *setting*, being that in which the mass loses its plasticity and becomes more or less friable; and the second, known as *hardening*, in which consolidation takes place, the mass gradually increasing in hardness with the passage of time, until a stony texture is obtained. It is probable that the reactions of setting involve the formation of super-saturated solutions, with deposition therefrom of close-knitted, interlacing crystals; while the slower process of hardening involves partly the formation of similar interlacing crystalline products, but more especially the production of a colloidal (amorphous) "gel," or glue, of calcium hydrosilicate, followed by gradual desiccation, or drying out. In the light of this knowledge, the grouping of cement particles and their maintenance in an unhydrated state, regardless of passage of time or surrounding conditions of moisture, will be better understood.

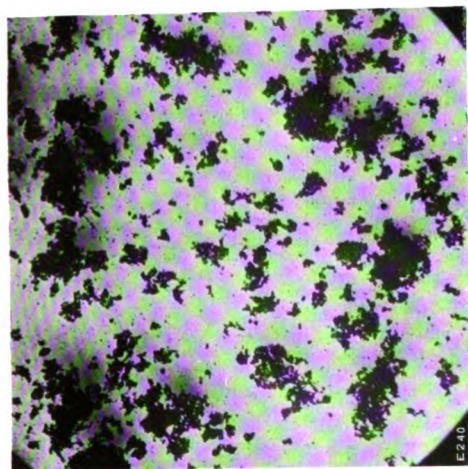
In Article 5 of this series the effect of surface tension, as exhibited by the close compacting of materials brought into contact with water, was demonstrated, such compacting being effective in direct proportion to the size and mass of the particles. With a material so fine as cement this compacting must be extreme. But close compacting of these particles would necessarily mean the exclusion of all but a very small percentage of water from between the particles; and this being so, it is beyond question that a highly supersaturated solution of the more soluble portions of the cement would speedily result, with consequent deposition in crystalline form of those highly-soluble aluminates, to which initial set is due. It should be understood further that under such conditions this process is completed in a far shorter time than is required to produce initial set where free access of water is permitted, so that in a period that is very probably measured in seconds the cement particles forming these groups are bound together closely by interlacing crystals, possibly of the same type as those in the photograph of *Fig. 31*, which shows the crystals formed by breathing on the surface of an unusually large piece of cement clinker, found in an old concrete taken from a dock wall in New York Harbour. But whether of this variety or not, since these groupings remain intact, this crystalline bond is evidently of a strength sufficient to withstand the mechanical agitations of mixing, especially when the mixing time is cut short in order to have the mixer deliver "a batch a minute." And as time goes on, the bond increases further in strength by the gradual formation of colloidal products, which desiccate through the absorption of water by the interior of the particles, producing those enduring honeycomb structures found in the cement matrix of all concretes, regardless of age.

INSUFFICIENCY OF WATER.

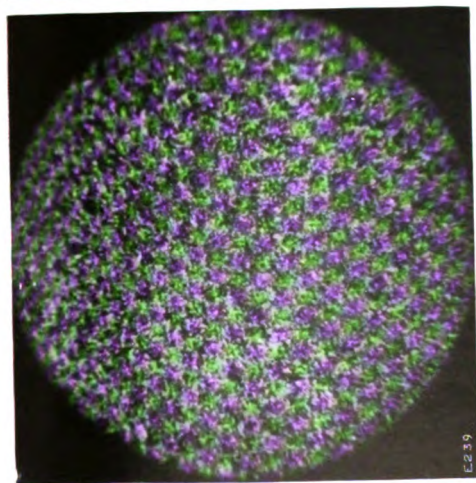
It should be recognised, also, that this colloid boundary, which entirely surrounds each particle, operates further in the prevention of progressive hydration by reason of its constituting an impervious envelope or skin, through which water cannot pass. As before stated, close grouping admits of the presence of only a small quantity of water between the particles, far too little to satisfy the chemical requirements of the cement; and since further water attack is prevented by this envelope, it should not



(c) After Addition of Alcohol to Water.



(b) Cement with Water Added, Deflocculator to Mixing Water Causes Dispersion of Cement Groups; 60 diameters.



(a) Dry Cement, Passing 100-Mesh Sieve.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

be a matter of surprise that concretes submerged for 30 years or more show about the same percentage of unhydrated cement as those quite recently made. Time and extraneous moisture are not the only factors necessary for effective hydration.

This outline of the formative processes of these groups of unhydrated particles is not merely an hypothesis. It is quite susceptible of visual demonstration, and, further, through such demonstration, remedial measures of the greatest significance are indicated.

In Fig. 32 at (a) is shown Portland cement of a fineness passing a 100-mesh sieve, dusted on a microscope slide. The magnification is low, but it is high enough to enable the individual particles to be distinguished. At (b) is shown the same field, after the addition of water to the cement. No longer are the particles individual, but grouped in close masses of varying sizes, whose general outlines are not unlike the outlines of the groups found in the standard sand mortar and in concretes. Obviously, if time were allowed sufficient for the primary crystalline and the later colloidal products to form between the previously individual particles which are known to compose these masses, their continuance in their present grouped and unhydrated state would be the natural expectation.

LOWERING SURFACE TENSION

If hydration is to be promoted, and if these groups may be held in a large part responsible for the low percentage of hydration, it is obviously necessary either to break them up, or to prevent their formation. Further, since *surface tension* seems to be initially responsible for their existence, the most obvious remedy would seem to be to lower the surface tension of the mixing water by one means or another.

In (c) of the same figure is seen the effect of doing this by an addition of alcohol to the water of the field seen in (b). The effect is immediate and instantaneous. *No longer is the cement massed in large groups, but is widely and generally dispersed.* In thus lowering the surface tension, those forces which before limited the boundaries

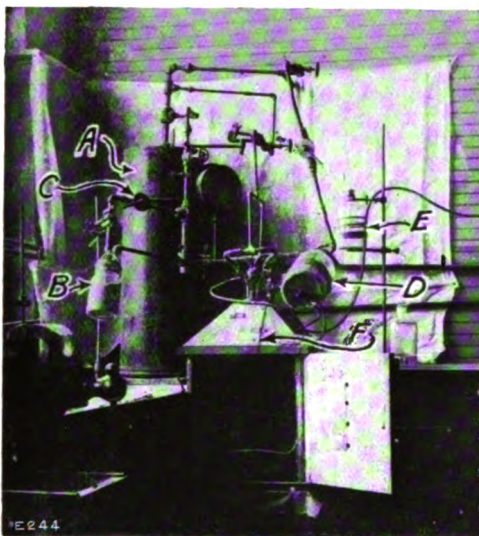


Fig. 33. Hydrating Apparatus (discarded).
THE MICROSCOPE IN THE STUDY OF CONCRETE.

of the water were so lowered as to extend the fluid, so that in this last photograph, only a portion of the material shown in the others remains in the field, but the effect on the groupings is none the less evident. Beyond any question, the particles in (c) are more individualised and in a far better situation to react with the mixing water than were those in (b); and if such a result were obtainable in commercial mixing, with a proportionate quantitative increase in hydration of cement, there should result a most important and desirable improvement in both the strength and quality of concrete. But before taking up a consideration of the results obtained from experiments along these lines, it is well to consider the wider import of such an improvement.

THEORETICAL LOSS DUE TO LOW PERCENTAGE OF HYDRATION.

There is no exaggeration in the statement that not more than 20 per cent. of the total cement added to concrete is hydrated, or usefully employed in any way save as a space-filler—a function better and more cheaply performed by a high-strength material, such as sand. On the contrary, there is already ample proof of this low efficiency of use derived not only from the researches of the writer, but from those of the Bureau of Standards at Pittsburgh as well, to which reference is made by permission. It further seems, as investigation is extended, as if this figure of 20 per cent. is conservative and that the actual percentage of hydration is below that figure by a considerable margin. But for the present, the percentage hydrated will be considered as 20 per cent. of the total cement.

It should be understood, however, that the inference is not to be taken that with complete hydration only one-fifth of the present quantity of cement would be required. Such an inference is not warranted by what is yet known of this material, but it is beyond question that with efficient hydration a very substantial reduction in quantity for any given strength could be made.

On this basis of 20 per cent., however, consider the economic loss involved. During the year 1913 the cement production of the United States, in round figures, was 92,000,000 barrels. Assuming an average price of \$1 per barrel, the total cost to the consumer for twelve months was \$92,000,000; and on the basis of 20 per cent. being usefully employed in concrete, the active value of the product was \$18,400,000, or a loss of \$73,600,000. To offset this would have been the value of the sand required to replace as a space-filler the 80 per cent. of unhydrated cement—in round figures, \$2,361,000, leaving as a direct net economic loss the stupendous sum of

\$71,239,000, chargeable apparently as wastage to the concrete industry. It behoves engineers to ponder these figures and to be alert to any means that promise their reduction

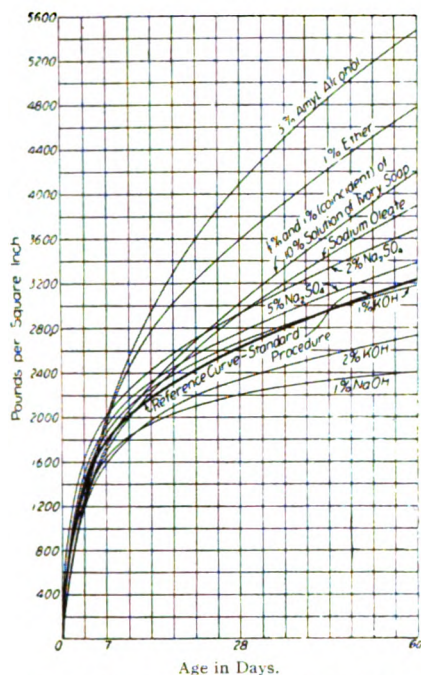


Fig. 34. Effect on Strength of Additions of Various Deflocculating Chemicals.

THE MICROSCOPE IN THE STUDY OF CONCRETE.

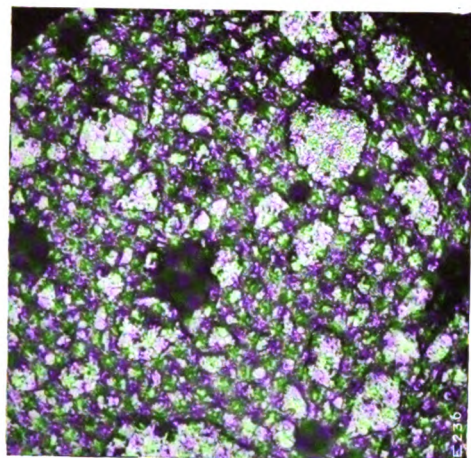
In this series it has been sought to emphasize the importance of other factors—*proper choice and proportioning of aggregates, sufficient mixing, proper consistency and care in placing*. There is more trouble traceable to neglect of these matters than to poor quality of cement; and although in this paper attention will be confined to the cement, these other factors should not be forgotten in considering the production of good concrete.

But effective hydration of cement is directly related to failure prevention. The removal of initial imperfections, such as voids, or poor contact of cement and aggregates, by swelling with hydration, is one of the most valuable properties of cement. The formation of gelatinous colloid by this process has before been noted. In its formation, large quantities of water are absorbed, but without net increase in bulk, as the swelled colloid occupies the space formerly taken up by the cement and water separately. If this should appear to be of minor importance, consider the point brought on in the paper preceding—that each 62½ lb. of water-weight in a cubic yard of concrete means one cubic foot of space occupied by water; and that loss of this water by evaporation and seepage means one cubic foot of voids remaining. Granted, however, the thorough hydration of a proper quantity of cement; not only would such voids be occupied by a dense, chemically-inert substance, not subject to the action of any natural water, but also their entrance, with resultant concentration of solution and disruptive crystallisation of their salts, would be prevented. And to touch upon a point not

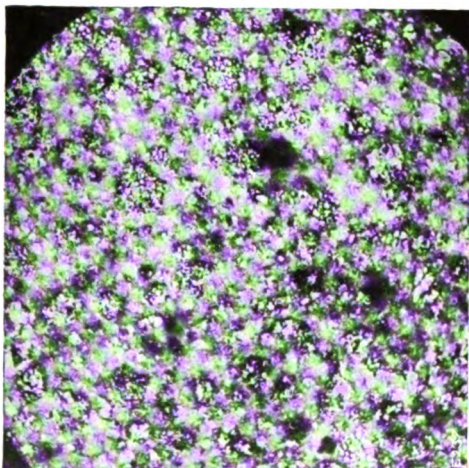
But this direct loss of \$71,239,000 is not the full tale. It is profitable to consider the value of those structures in which this cement was incorporated. It is an acknowledged, indisputable fact that failures of concrete structures have occurred in the past for one reason and another; and it is reasonable to assume that other failures will occur in the future. In a previous article of this series (see Article 2, July issue), some of the causes of such failure have been demonstrated; and it has been shown that certain of these causes of failure have their inception in these imperfections resulting from directly assignable causes in mixing and placing. How much of the 1913 cement is in concrete of this character? The question is impossible to answer, but reasoning from past experience and considering the limited life of such structures and incidental losses, it is probably not an exaggeration to say that with passage of years the original figures of direct loss will be duplicated.

HYDRATION AND FAILURES.

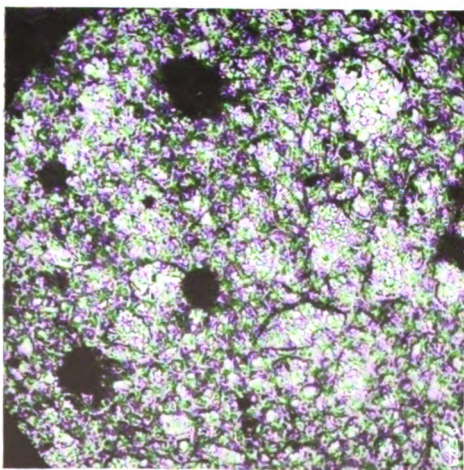
It should be understood, however, that hydration, or the lack of it, should not be held solely responsible for the production of poor concrete, with possible resultant failure.



(c) Strength at 28 Days, 715 lb.
50 diameters.



(b) Strength at 28 Days, 640 lb.



(a) Strength at 28 Days, 772 lb.

Photo-Micrographs show Character of Matrix of Neat Briquettes Tested in Tension ; 50 diameters.
THE MICROSCOPE IN THE STUDY OF CONCRETE.

previously considered, but one of the highest importance, *the fire-proofing value of concrete is a direct function of the degree of hydration.* Extended experiments by C. W. Hunt at Sibley College in 1912 showed that the more effective the hydration—that is, the greater the colloid content in the cement mass, the lower the coefficient of transmission of heat. Further experiments are needed along this line.

EXPERIMENTS TO INCREASE STRENGTH.

The property of strength, however, is that of most importance and interest to users of concrete. It is also the most easily determinable of all the properties of materials and the easiest of useful employment in the arts. Increasing the strength, tensile and compressive, was the primary object of this research; and before the nature of the lumps of cement found in mortars and concretes by microscopic investigation was known, it was believed that increasing the colloid through increasing the effective of hydration was the proper method of procedure.

To this end it was first sought to secure intimate contact between cement and water by means of pressure, as in a wringer. This proved quite ineffectual. Drawing water through the mass by a vacuum was next tried, with the same result. Finally, after much experimenting, the apparatus shown in Fig. 33 was devised in 1912, with some rather successful results. Essentially it consisted of a container (B), from which cement was delivered by an air-boil, taking its air from the expansion tank (A); a water supply (E); a sand supply (D), delivered by a similar air-boil; a wooden receiver (F); and in the top of this receiver a nozzle, composed of a central

delivery pipe for sand and cement, while surrounding it were eight throat atomisers, to finely divide the water supplied. It was believed that by these means each cement particle would, in transit through the receiver, obtain its mating water-particle, with consequent promotion of hydration. A pan at the bottom of the receiver caught the wetted mortar.

MECHANICAL MEANS OF DISPERSION ABANDONED.

Although decidedly crude in many ways, this apparatus was quite successful in operation, results being achieved by it that subsequent machines, more mechanically perfect and supposedly identical in operation, failed to secure on a larger scale. Although for a long time provocative of much labour and discouragement, it is now known that coalescing of water-drops, resulting from a multiplication of nozzles, intensified the surface tension effects, thus producing that grouping of cement particles which it was sought to avoid. In consequence, plans for mechanical dispersion had to be abandoned.

Many things appear simple in retrospect which were exceedingly complex during the period of their development. The determination of surface-tension as the dominating factor in preventing free access of water to cement required a protracted period of intensive study, but once this point was cleared up, the securing of more effective hydration became a matter of relative simplicity.

DISPERSIVE EFFECTS OF CHEMICALS.

The dispersive effect of alcohol on groupings of cement particles has already been shown. Alcohol is, of course, not commercially applicable in the mixing of concrete, but it is not the only agent that might be employed. On the contrary, there are many such. Ether—also not commercial—is quite effective. Amyl. alcohol, though disagreeable to use, due to its unpleasant vapour, gives markedly superior results. A small percentage of a solution of ivory soap is of considerable assistance, and among other materials that have experimentally aided may be mentioned sodium oleate, sodium hydroxide, potassium hydroxide, and sodium carbonate (washing soda) and—strange as it may seem, sodium sulphate—that arch-enemy of sea-water concretes.

The use of these sodium alkalis and soaps was first suggested by a consideration of the density of those earths in which they are present in considerable proportion. It is not improbable that their influence as deflocculators has had an effect in the normal deposition of these earths. It is also interesting to note that the same agents are used in the preparation of clays for pottery work, where it is desired to have an evenly-wetted paste—an operation physically identical with the wetting of cement, but favoured by the absence of chemical reactions that cause the binding together of particles by intercrystalline formations. There are doubtless a host of other materials which would serve the same purposes, but these mentioned are those on which experiments have so far been conducted.

Preliminary results obtained by their use are plotted in *Fig. 34*. These are interesting and promising, indicating what may be accomplished once the right proportion of the proper additive is discovered. Of necessity, it must be cheap in first cost and of a nature such that no detrimental effect shall be exercised through its agency. It is desired in this paper to present tentative results on mortars only, reserving longer-time experiments on concretes of varying proportions for later publication. But in view of the seeming correctness of the reasoning on which their conduct was predicated, and of the confirmation these favourable results afford, they are made public at this time in order that as wide an appeal as possible may be made for co-operation along these lines by all who are interested in the betterment of concrete and the growth of the industry. The subject is too vast and of too great import to be developed ade-

quately by a single experimenter. The secret of accomplishment lies in organised effort by many, in order that inadvertent individual errors may be eliminated and the truth ascertained. It may be that rich rewards will fall to the lot of some fortunate investigator who shall perfect a patentable process, but for the present the question belongs to the world, and such information as is at hand is freely given.

SURFACE TENSION LOWERED BY HOT WATER.

But perhaps most interesting of all are the results obtained by the use of *hot water* in mixing concrete. Surface tension is a function of boiling point. A liquid of low boiling point, like alcohol or ether, has a low surface tension. In the same way a liquid such as water, when elevated in temperature near to its boiling point, has a low surface tension. Furthermore, chemical reactions are speeded up, approximately as the twentieth power of their absolute temperature—a fact finding its expression in the subject in hand by the almost instantaneous reaction and formation of crystalline compounds, resulting, as is so well known, in immediate set, when cement is gauged with hot water. Expressed in tabular form, and taking 60 deg. Fahr. as a base, the time required for chemical reaction varies with temperature as follows:—

Temp. Fahr.	Time Required for Reaction.
60 degrees	1 (base)
70 "	0'69
100 "	0'23
150 "	0'049
210 "	0'0062

If, therefore, by some means the formation of initial set could be prevented, gauging the cement with water at 210 deg. would not only result in a reaction 16,000 times as fast as that obtainable at 60 deg. Fahr., but would have the further advantage of reducing the surface tension factor so low as to be highly advantageous. Practically this instant set can be prevented by mechanical stirring, and the effect of these hot-water additions as exhibited by photomicrographs and by physical tests are shown, as in *Fig. 35*.

Although it is desired to warn against unwarranted conclusions as regards degree of hydration being drawn from these photomicrographs, the appearance of (*b*) mixed with water at 150 deg., age of specimen twenty-eight days, is especially pleasing in the apparently large colloidal content, as shown by the prevalence of structureless white masses. It should be further noted that in this photograph the voids are relatively smaller than in the others and of uncertain outline, indicating the gradual filling of pores by colloid in the manner before indicated.

In interpreting these photographs the relative percentage of cement-particle groupings is first to be noted. In (*a*) where the mixing water was at 100 deg. Fahr., these groupings are quite extreme. Evidently the surface tension was not sufficiently lowered by this rise of temperature to offset the accelerated chemical reactions producing the interlacing of grouped particles. The tensile strength of this briquette at twenty-eight days was 772 lb. per sq. in.

With the mixing water at 150 deg., the grouping is a minimum, with prevalence of white, cloudy patches, indicating a large percentage of flocculent colloid, and confirmatory of this is the lower tensile strength of 640 lb. per sq. in., for the strength of colloid is not developed until the passage of considerable time, while the strength of crystalloids, as in the groups shown at (*a*), is developed almost instantaneously.

In (*c*) is shown a briquette made with the mixing water at 200 deg. Fahr. The

groupings are in goodly number, though not excessive; and the colloid content is probably larger than in (a), for the tensile strength is 715 lb. per sq. in.—midway between the others. There is very probably a critical temperature at which the best results would be obtained, but what this temperature is has not yet been determined. It is, of course, possible that the differences here shown are attributable to manipulation, but as all briquettes were made by the same operative this possibility is assumed to be negligible.

These experiments and their results seem to find confirmation in the superior quality of concrete often made from heated aggregates in cold weather. A number of observers have noted this.

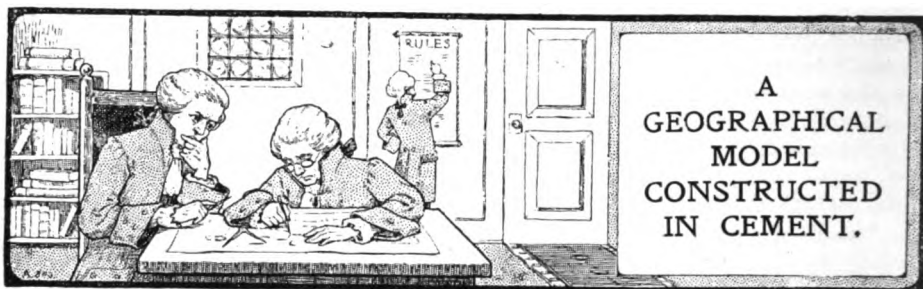
WETTING SURFACE OF AGGREGATE.

There is also another highly desirable result attainable through lowering the surface tension of mixing water. The reluctance of water to wet a dry surface, such as that of aggregate, is well known. Liquids of low surface tension, such as alcohol, or water containing such substances as the sodium alkali used in the mortar experiments before noted, do not exhibit this reluctance, but, on the contrary, pass freely over a dry surface with eagerness, as though there were affinity between them.

In the paper on the proportioning of aggregate (see Article 3, August issue), it was pointed out that where pieces of large aggregates had been dislodged from their bedding in the mortar of 8-in. sections, there was every evidence that thousands of air bells, if not an almost complete envelope of air, had clung to the surface of the aggregate, preventing contact between cement and aggregate; and it is to be inferred from such instances that similar conditions obtain throughout the mass. In micro-sections (as in Article 1, June issue) the same conditions were shown to obtain with a sand grain; and it requires but little thought for anyone to realise that this occlusion of air is another manifestation of the effect of surface tension of water; and corollary to this that the reduction of the mechanical value of this surface tension is one way, if not the only way, of overcoming such difficulty. Further, since these air bells and air films occupy space in the mass, it follows that their removal would strengthen concrete, not only by replacing air with material of high strength, but also because such removal would permit a closer compacting of aggregates, with all the consequent advantages accruing to the production of a substance having a density approaching more nearly that of natural stone. Just as defects are cumulative in their pernicious effect, improvements are in like measure valuable. Concrete is a composite substance; and only by having regard for the natural properties of its component materials, not the least important of which is water, may proper results be obtained.

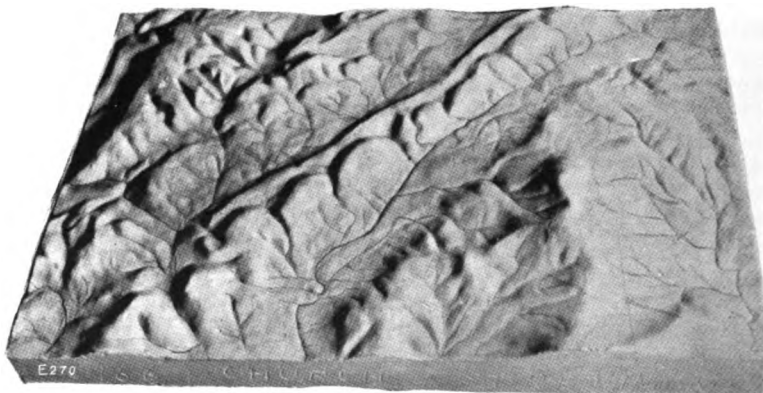
WIDER STUDY NEEDED.

In the six papers of this series it has been sought to place before the engineering world at large certain considerations, old and new, in regard to concrete in such sequence and relation as to show their importance, not only individually, but collectively as well. Necessarily the treatment of each phase of the question has been brief, but it is hoped that some suggestions have been made such that further investigations as to the undeveloped possibilities of concrete will be carried out by others.



The following particulars are an example of the use of cement for model work.—ED.

THERE has recently been added to the treasures of the National Museum of Wales a relief model of the Principality of Wales. This map, which took many years to complete, is made in cement, and consists of sixty-seven separate blocks. It is on a scale of 1 in. to the mile. The surface has been covered with white enamel paint, and the streams, canals, and lakes are painted blue, whilst the sea is enamelled rather lighter in tint. Nearly half a ton of cement has been used for the whole map, and the separate blocks average about 16 lb. The



Relief Model of Wales: Church Stretton.
CEMENT FOR GEOGRAPHICAL RELIEF MODEL.

authors are under the impression that this is the largest model of its kind and covering so great an area. It measures 12 ft. by 10 ft. 6 ins. over all.

In a very interesting handbook, published by the Museum at Cardiff and written by Mr. Wallace Whitehouse, L.C.P., on the model, some details are given of the method of construction, which was fraught with many difficulties. We give briefly a few notes taken from this book.

After the preparation of the Ordnance maps and cards employed the model was first prepared in plasticine, but this was found unsuitable for permanent use. Plaster of Paris was then resorted to, but this, owing to its fragile nature and quick setting qualities, was replaced by cement, which proved eminently suitable.

The plasticine models were placed in a specially constructed casting box. The inside of the casting box and the plasticine model were oiled, and some 16 lb. of cement mixed to a thick cream-like consistency were poured in. This

gave a cement negative generally about $1\frac{3}{4}$ in. thick, which could be removed from the casting box in five to nine hours.

This negative mould was examined, all pin holes were filled, and the surface painted as indicated above. This operation was repeated for each block, and from these negatives cement positives were cast.

It was found that cement, carefully mixed, gave a very convenient medium

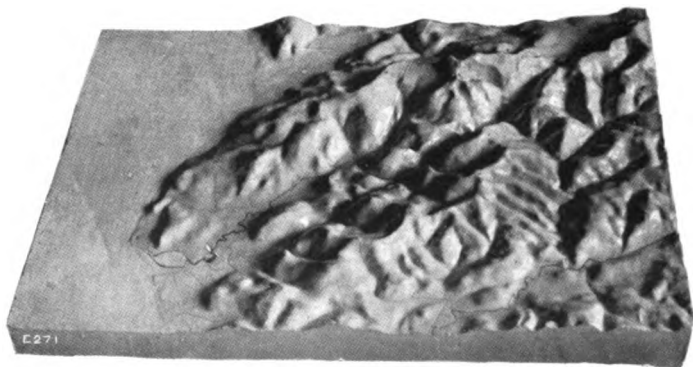


Relief Model of Wales: Haverfordwest.
CEMENT FOR GEOGRAPHICAL RELIEF MODEL.

for plastic modelling. In several cases quite extensive repairs to models had to be carried out, and the added portion is in every case firmly joined to the old mass and is similar in appearance.

We give herewith three illustrations of finished blocks of the model.

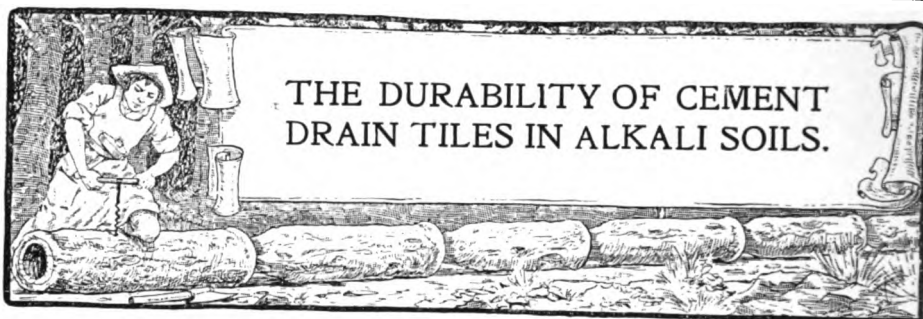
The map was mainly the work of Mr. Wallace E. Whitehouse, Assistant Lecturer in Physical Geography at University College of Wales, Aberystwyth, who was responsible for much of the modelling, the whole of the casting, and



Relief Model of Wales: Barmouth.
CEMENT FOR GEOGRAPHICAL RELIEF MODEL.

all checking and revision, though Dr. H. J. Fleure was responsible for the inception of the scheme. A number of post-graduate students co-operated in the first three years, whilst much assistance was also rendered by Mr. F. S. Wright.

The handbook also analyses the relief of Wales with regard to its geology, and discusses drainage and human geography.



THE DURABILITY OF CEMENT DRAIN TILES IN ALKALI SOILS.

By R. J. WIG and G. M. WILLIAMS.

(*Technologic Papers of the U. S. Bureau of Standards, No. 44.*)

Some interesting investigations have been carried out on the above subject and reported upon by The Bureau of Standards, U.S.A. We give below a short résumé of the Report, from which we have also reproduced our two illustrations.—ED.

THE disintegration of concrete when exposed to water strongly charged with salts of the alkalis, as in the arid regions of the Western United States, has been a source of great trouble to engineers. Sometimes, however, defects are attributed to alkali salts for which the bad composition or mixing of the concrete is responsible.

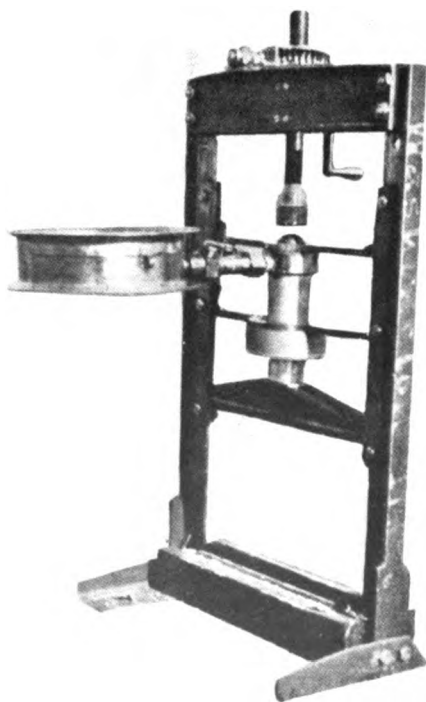


Fig. 1. Portable Tile-Testing Machine of three-edge bearing type, used in testing experimental tiles at the site of the drain.

THE DURABILITY OF CEMENT DRAIN TILES IN ALKALI SOILS.

There are two ways in which disintegration may be brought about: by crystallisation of salts in the pores, mechanically breaking up the mass, and by chemical action on the lime of the cement. Laboratory tests show that all mortars may be made to disintegrate by means of alkali salts, and the present investigation has therefore been made in the field.

The cement drain-pipes used for the test are mostly machine made, an Iowa brand of Portland cement being mixed with a sand, also from Iowa, some experiments also being made with a sand cement manufactured in Idaho by the U.S. Reclamation Service. The usual machine-made pipes (or drain tiles) are $12\frac{1}{2}$ in. long and $\frac{3}{4}$ in. thick, with an inside diameter of 8 in., and the hand-made tiles have the same length and inside diameter, but are $1\frac{1}{2}$ in. thick, and are made in sheet-iron moulds. Different mortars have been tested, and also the effect of different degrees of plasticity. The standard commercial machine-made tile is composed of 1 part of cement by weight to 4 parts sand, of plastic consistence, cured in steam. The sand-cement tiles have a chalky appearance. Sites in Wyoming, Montana, Utah, Arizona, New Mexico, Colorado, Iowa, and Minnesota have been selected for the tests.

Crushing tests of the pipes are made by means of a portable machine, three edges being used for the bearing, two below and one above (*Fig. 1*). The pipes, after remaining the prescribed time in the soil, are uncovered, one or two at a time, examined, measured, and then tested in the press, care being taken that they are placed in the press in the same position in which they rested in the soil. The time of exposure in all the tests was one year, and some of the localities chosen showed a remarkably high concentration of alkali salts, the ground being encrusted with a thick white deposit.

The general results indicate that most of the tiles are uninjured in strength by exposure to these severe conditions. At three of the sites none of the tiles showed any abnormal change. At another, only the poorest mixtures were found to have deteriorated. Low results were obtained for some of the hand-made tiles, evidently on account of defective tamping. The effect of alkali salts on a lean mortar is seen in *Fig. 2*. This represents one of the ordinary machine-made tiles, 1:4 cement and sand by weight, after a year in soil at Grand Junction, Colorado, where the land had been cultivated and afterwards abandoned on account of the alkalinity of the soil. At the time of beginning the experiment the ground was thickly covered with a white coating of salts, and there was no vegetation except alkali weed.

The amount of absorption of water by the tiles showed great and somewhat erratic variations. Thus the highest absorption was given by tiles coated with tar and by others which had been treated with ferrous sulphate as a means of diminishing the porosity. No explanation of this behaviour has been given. The absorption of machine-made tiles is in general the greater, the poorer the mixture employed. High porosity is naturally also a danger in regard to the chemical action of the salts.

The principal conclusions of the Report may be summarised as follows: Only the best materials and good workmanship are permissible in the preparation of cement drain tiles for use in highly alkaline soils. Failing these, serious

disintegration is likely to take place. Cement mortars which contain not less than one part of cement to three parts of aggregate are almost unaffected by one year's exposure to the conditions of these tests. Poorer mixtures should not be used where the concentration of alkali is high. It is not possible to



Fig. 2. Swollen and Cracked Tile after 1 year's exposure. The interior surface is cracked similar to the exterior
THE DURABILITY OF CEMENT DRAIN TILES IN ALKALI SOIL.

draw further general conclusions until the range of the experiments has been extended.

The authors ask for information as to the behaviour of cement or concrete exposed to the action of alkalis in cases which have not come under their notice.



It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.—ED.

U.S. DEPARTMENT OF AGRICULTURE.
PORTLAND CEMENT CONCRETE PAVEMENTS
FOR COUNTRY ROADS.

By **CHARLES H. MOOREFIELD** and **JAMES T. VOSHELL,**
Senior Highway Engineers.

The following is an abstract from Bulletin No. 249 of the United States Department of Agriculture.

INTRODUCTION.

It is realised that the present state of knowledge concerning the best methods of constructing such pavements is by no means complete, and those who have charge of concrete road construction should be careful to keep themselves informed regarding results obtained by others engaged in similar work and by laboratory experiments.

MATERIALS.

No hard and fast rules can be laid down which would fit all cases in the selection of concrete materials, as availability is necessarily a very important factor.

Cement.—Satisfactory cement can usually be obtained, and none should be used in constructing pavements which does not meet all the requirements for a high-grade Portland cement.

Sand.—Sand for use in concrete pavements should be selected with especial care. Preference should be given to sand composed of a mixture of coarse and fine grains, with the coarse grains predominating, though sand consisting entirely of coarse grains is preferable to that in which the fine grains predominate. It is also very important that the sand be as clean as practicable. Sand which contains more than about 3 per cent. of foreign materials, such as loam or clay, should be rejected, and no sand should be used the grains of which are coated with clay or other objectionable material.

Sand which contains even a very small percentage of vegetable acids is unsuitable for use in concrete, because such acids seriously affect the strength of cement.

Coarse Aggregate.—There are not sufficient data available to warrant making a definite comparison of the advantages possessed by the different varieties of stone when used as coarse aggregate. But so far as cracks are concerned limestone appears to have made a better record than any other variety of stone which has been used to any considerable extent.

The coarse aggregate, whether of crushed stone or gravel, should possess at least as great resistance to wear as the mortar which fills the voids between the particles of stone.

It is very desirable that the particles composing the coarse aggregate be well graded in size between proper limits in order that the percentage of voids may be as small as practicable.

Water.—Water used in mixing concrete should be reasonably clear and free from alkalis, acids, vegetable matter, or other injurious materials.

Proportioning.—Concrete in pavements is subjected to much more severe service conditions than that in walls, foundations, etc. Most of the old rules for proportioning concrete were developed with a view to providing only for simple compressive stresses, such as are met with in the latter class of structures. Hence it is not surprising that the early results obtained for pavements by following the old rules were not generally satisfactory. Concrete pavements must resist not only crushing and impact stresses, but the wearing action of traffic as well, and this is probably the most destructive process to which they are subjected.

The essential qualities which enable any material to withstand the wearing action of traffic are hardness and toughness. Laboratory tests have been devised for determining the relative degree in which these qualities are possessed by different kinds of stone and brick, but none of these tests is suitable for making similar determinations regarding concrete mixed in different proportions and composed of different materials. The proper proportion in which to mix the materials can probably be best determined from actual service tests.

CONSTRUCTION.

Types.—There are two general types of concrete pavement, known as the one course and the two course. These designations are due to the fact that the former consists of one course of concrete, all of which is mixed in the same proportion and composed of the same kind of materials, while the latter consists of two courses of concrete, usually mixed in different proportions and containing different kinds of aggregate. On page 634, *Fig. 1*, will be found a typical cross section for a concrete pavement, and this general form is suitable for either one-course or two-course work. The one-course pavement is somewhat simpler to construct than the two-course type. It possesses the advantages that there is no possibility for the wearing surface to separate from the rest of the pavement, and that the resistance to wear should be uniform throughout the life of the pavement. Notwithstanding these advantages, local conditions may sometimes justify the two-course type of construction. For example, if the old materials locally available for use as aggregate were of very inferior quality, it might be more economical to use them for aggregate in the lower course of a two-course pavement and import aggregate for the wearing course than to employ a one-course pavement and import all the aggregate.

The one-course pavement is believed to be better adapted to most ordinary conditions than any other type of concrete pavement.

Grading and Preparing the Subgrade.—In forming a roadbed upon which a concrete pavement is to be constructed the features which should receive primary consideration are (1) adequate drainage, (2) firmness, and (3) uniformity in grade and cross section.

It is impracticable to prescribe definite methods for securing thorough drainage which would be applicable to every location. It is only by means of a careful study of local conditions that a satisfactory system of drainage can be devised.

The second requirement, firmness, can be secured only after the road has been properly drained. Soils which readily absorb moisture will not remain firm in wet weather, and therefore should not be permitted to form a part of the roadbed, especially if they occur in the subgrade. This requirement also makes it necessary that the roadbed be thoroughly compacted. In forming embankments the material should be put down in layers not more than about 12 in. thick, and each layer should be thoroughly rolled. The subgrade in both excavation and embankment should be brought to its final shape by means of picks and shovels and rolling.

The cross section of the subgrade may be either flat or shaped to conform with the finished surface of the pavement. The flat cross section involves the use of a slight additional quantity of concrete, but gives an increased thickness at the centre, where maximum strength is required. It has been observed that longitudinal cracks occur less frequently in concrete pavements laid on a flat subgrade than where the subgrade is curved to conform to the surface of a crowned pavement.

In either case the subgrade when completed should be uniform in grade, cross section, and firmness, not only to prevent a waste of concrete in filling up depressions, but in order to facilitate the necessary movement of the pavement due to contraction and expansion and thus reduce its tendency to crack. The subgrade should be rolled and re-shaped until the specified shape is secured. The forms, which should be set before the final shaping, may be made to serve as a guide for this work.

Use of Sub-base.—Where old pavements which have been constructed on a sub-base are replaced by concrete pavements it is frequently convenient to place the new pavements on the old sub-base. Furthermore, soil conditions are sometimes such as to make the use of a sub-base very desirable. This is especially true of soils which do not compact readily under the roller or which cannot be effectively drained at a reasonable cost.

A satisfactory sub-base may be constructed of gravel, broken stone, telford, cinders, or any other similar material. The essential features in every case are firmness, smoothness, and uniformity in grade and cross section.

It has been suggested, with an apparent show of reason, that a thin cushion of sand might be advantageously used under concrete pavements. The purpose of this construction is to facilitate the sliding of the pavement, due to expansion and contraction, and thus to increase the allowable distance between contraction joints. So far as is known there are no experimental data which bear on this subject.

Forms.—The form work required for concrete pavements is very simple and inexpensive. Ordinarily the forms may consist of 2½-in. boards having a width equal to the edge thickness of the pavement, though metal forms are in general more economical and are always to be preferred. The forms should be set before the subgrade is finished, in order to serve as a guide for the finish grading, and should be securely held in place by means of stakes driven on the shoulder side to such depth that they do not extend above the top of the forms.

The forms should always be set true to line and grade, and where curbs or gutters are to be provided they must be modified to suit the requirements for these features.

Mixing and Placing the Concrete.—When a considerable area of concrete pavement is to be laid it is usually economical to employ a mechanical mixer for mixing the concrete. Hand mixing is much more expensive than machine mixing, and hand-mixed concrete is rarely as uniform as machine-mixed concrete either in consistency or in the distribution of the component materials. Since lack of uniformity is believed to be one of the most potent causes for the formation of cracks, machine mixing is greatly to be preferred.

Even when the very best type of concrete mixer is employed it is necessary to exercise considerable care to see that the concrete is mixed thoroughly and to a uniform consistency. Tests have shown that increasing the time during which a batch of concrete remains in the revolving drum of a mixer, within reasonable limits, has very much the same effect as increasing the proportion of cement. It is also almost certain that varying amounts of water in successive batches will tend to cause cracks to develop in the pavement. It is impracticable to state definite rules for determining the number of turns of the mixer drum or the exact quantity of water which each batch should be given. In general it may be said that each batch should be mixed until there are no uncoated particles of sand or coarse aggregate remaining, and the amount of water should be such that the resulting concrete will be quaky or jellylike, but not sufficiently wet to flow readily while it is being handled. On steep grades somewhat less water should be used in mixing the concrete than when the grade is level.

Immediately after the concrete is mixed it should be deposited in the pavement.

Before any concrete is placed the subgrade should be thoroughly sprinkled with water or a part of the water contained in the concrete will be absorbed by the subgrade, which may interfere with the process of setting.

For one-course work the concrete should be deposited between the forms in such quantity that when it is struck off and compacted it will present a uniform surface and have the depth required for the finished pavement. Each batch of concrete should be dumped as nearly in place as is practicable and should preferably be spread by means of mortar hoes. The men who do the spreading should avoid walking in the concrete.

After the concrete has been spread approximately to the required cross section it should be struck off with a strike board having slightly more crown than the cross section of the road. This allows for a slight amount of settlement when the concrete is compacted. The compacting should be done with a tamper shaped to conform with the cross section of the road and operated by two men, one standing on each side of the pavement. Suitable designs for strike boards are shown in *Figs. 2 and 3*, page 634.

Sometimes the tamping and striking off are done with the same template, but this is not altogether satisfactory, because when this is done it is impracticable for the template

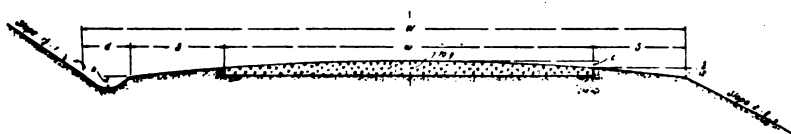


FIG. 1.—TYPICAL SECTION OF CONCRETE ROADWAY.

Side ditches should be of sufficient size to dispose of all drainage, C may vary from $\frac{1}{8}$ to $\frac{1}{4}$, when w exceeds 20 feet make joint in center and crown subgrade; k varies from 6 to 12 inches.

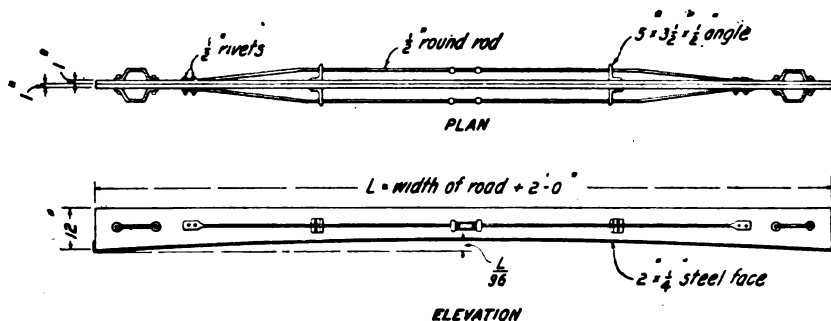


FIG. 2.—TYPICAL DESIGN FOR STRIKE BOARD

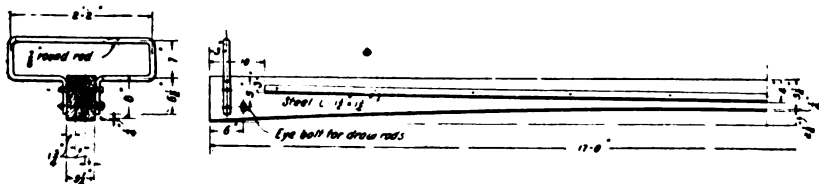


FIG. 3.—WOODEN STRIKE BOARD

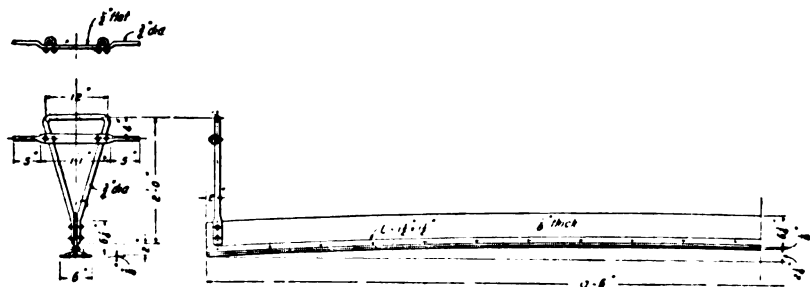


FIG. 4.—STEEL TAMPER

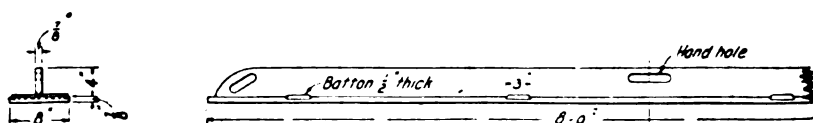


FIG. 5.—LONG WOODEN FLOAT.

to have a greater crown than is required for the finished pavement, and it is difficult to strike off the concrete with such a template and at the same time make provision for compacting.

In the case of two-course pavements it is important that the top course be placed before the concrete in the bottom course has taken its initial set. The bottom course should be well compacted and struck off, but the striking off need not be as carefully done as in the case of the top course. The top course should be constructed in a manner similar to that described for one-course pavements.

Finishing the Surface.—The surface of a concrete pavement may be given either a rough or a smooth finish. A slightly roughened surface has the advantage of being less slippery when the pavement is first constructed and is preferred by some engineers on that account. Smooth surfaces are more generally preferred, except on very steep grades, where it is sometimes desirable to provide grooves or other comparatively deep markings at right angles to the direction of traffic in order to afford a better foothold for horses. Such grooves, however, will cause rapid deterioration of the pavement under heavy traffic.

A satisfactory method of finishing the surface is to use a wooden float for smoothing out all template markings and evening up other slight irregularities. This method of finishing produces a surface sufficiently rough for all ordinary grades and possesses the advantage of being extremely simple. In using the float special care must be exercised to keep the pressure of the hand uniform, in order not to produce irregularities in the surface. Wherever a depression occurs it should be filled by adding concrete, and not by raking mortar into it with the float. The workmen who do the floating should be provided with one or more light bridges, which span the pavement and which can be easily moved as the work progresses. A long float, as shown in *Fig. 5*, page 634, requires less skill on the part of the operators than short floats. A suitable design for a finishing bridge is shown in *Fig. 1* on page 637.

Joints.—It is customary to provide transverse joints at regular intervals in concrete pavements, to prevent irregular cracks from being produced; and if the width of the pavement exceeds 20 ft. longitudinal joints are also usually provided.

If no transverse joints are constructed in the pavement, the length of the sections between cracks, judging from such limited data as are at present available, will vary from 20 to 150 ft., and depends upon the kind of aggregate used, the relative richness of the concrete, the condition of the subgrade at the time the concrete is placed upon it, and the method employed in curing the concrete. It is common practice to space the transverse joints from 25 to 50 ft.

If there were no curves in the alignment, or summits in the grade of a road, it is doubtful if any provision for expansion would be necessary in constructing the joints, because the elasticity of the concrete should be sufficient to take care of the expansion caused by changes in temperature and moisture content.

There are numbers of different methods of constructing joints, but none of them appears to be entirely satisfactory from every standpoint. Probably the simplest type of joint is that made by introducing into the pavement a board about five-eighths inch thick and shaped to conform with the cross section. This board is held in place by means of stakes until the concrete is placed against it on both sides. The stakes are then removed and the board is left in place with its upper edge even with the surface of the pavement and its lower edge resting upon the subgrade. The principal objections to this joint are that the board wears rather rapidly and does not protect the adjacent edges of the concrete.

The method most often used in constructing joints is to separate the successive sections of the pavement by means of specially prepared bituminous felt boards. These are usually held in place by means of properly shaped steel templates until the concrete is deposited against them, after which the templates are removed and the concrete flows around the boards. The thickness of this joint has varied in common practice from one thickness of two-ply tar paper up to about one-half inch. A thickness of one-quarter inch seems to give very satisfactory results when the joints are spaced about 30 ft. apart. Joints of this kind are sometimes provided with metal armour, which is intended to keep the adjacent edges of the concrete from being spalled off. It is claimed that armoured joints require less maintenance than other types, but they are more expensive to construct.

The joints are undoubtedly the weakest feature of the concrete pavement; and, no matter what type of joint is used, they must be given frequent and careful attention to prevent rapid deterioration of the pavement adjacent to them.

In the past, contraction joints of all types have usually been placed at right angles to the line of the pavement. This method of construction has the disadvantage that two wheels of a vehicle strike the joint at the same time and thus produce the maximum amount of impact. By skewing the joint at an angle of about 15 degrees the wheels strike one at a time, and the total resultant impact is reduced by at least one-half. This is advantageous to both the traffic and the pavement, and, since the difficulties involved in constructing skewed joints are not at all serious, there is no apparent objection to their use.

Protecting and Curing the Concrete.—The quality of the concrete depends to a great extent upon the conditions under which it sets or hardens. When early exposed to dry air, for example, water is evaporated out, thereby greatly accelerating the shrinkage of the concrete and delaying the process of setting. It is evident that these results form a very effective combination for producing cracks. The effect of freezing on concrete is still more harmful; not only are cracks produced, but the internal structure of the concrete is also damaged.

The precautions that must be taken in order to protect a newly-constructed concrete pavement during the process of curing depend largely on the weather conditions.

The protection of the concrete is an extremely important feature of concrete pavement construction. It is impossible to secure satisfactory results unless some precautions are taken to prevent the concrete from drying out too rapidly after it is placed, and to ensure that it sets up under uniformly favourable conditions.

The Use of Reinforcing Steel.—Probably the most satisfactory method, in point of efficiency, yet devised for reducing the number of objectionable cracks in concrete pavements is that of employing steel reinforcement. The reinforcement usually consists of woven wire or some similar material, though there is no apparent reason why plain round or square rods might not be satisfactorily used. One-quarter-inch round rods embedded about 2 in. above the lower surface of the pavement and spaced about 12 in. centre to centre in both directions would seem sufficient to eliminate practically all objectionable cracking, provided proper joints were introduced at changes in the grade and at curves in the alignment. But any satisfactory system of reinforcement will probably add from 15 to 20 cents per sq. yd. to the cost of the pavement, and this additional cost is no doubt responsible for the fact that concrete pavements are seldom reinforced. Furthermore, reinforced pavements are more difficult to repair than those made of plain concrete, which may be a serious objection under some circumstances.

Gutters.—It is frequently desirable to provide concrete pavements with paved gutters in order to prevent the side ditches from eroding. Fig. 2, page 637, shows a typical design for a concrete gutter. This design has been frequently used and has usually proved to be satisfactory. A suitable strike board for forming this gutter is also shown in the figure.

It is impracticable to construct the pavement and the gutter at the same time, and on account of the convenience of using the pavement as a platform for material and for mixing concrete for the gutter the pavement is usually constructed first. When there is no space between the gutter and pavement the joints should always be continued through both. If this is not done the joints in each are apt to be continued as cracks in the other.

Curbs.—Concrete pavements on country roads are not generally provided with curbs, because it is usually desirable to use the shoulders as part of the roadway. Under some circumstances, however, curbs may be employed to advantage. For example, in deep cuts it might be justifiable economy to omit the shoulders and side ditches and provide curbs along the edges of the pavement so that the sides of the pavement would serve as gutters. Likewise on very deep fills curbs are sometimes used to protect slopes from erosion. When this is done it is necessary to provide catch basins at low points in the grade.

Bituminous Wearing Surface.—Various kinds of bituminous materials have been used and several methods of applying them have been tried. Some of the surfaces are reported to have given moderately good service under light traffic, but in general

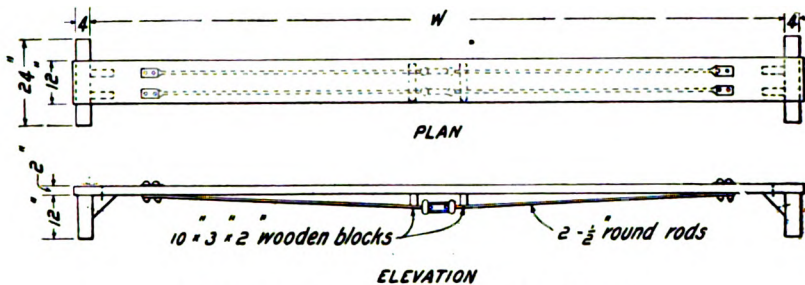


FIG. 1.—TYPICAL DESIGN FOR FINISHER'S BRIDGE

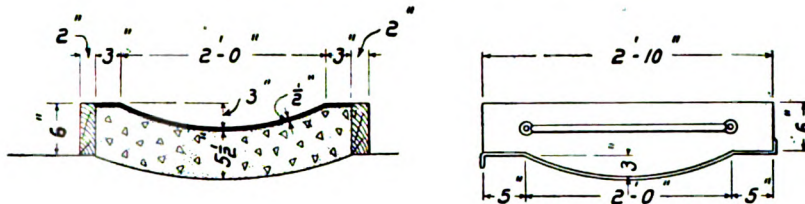


FIG. 2.—TYPICAL CROSS SECTION OF CONCRETE GUTTER AND DESIGN FOR A TEMPLATE TO BE USED IN ITS CONSTRUCTION

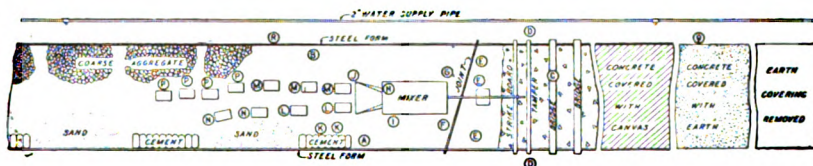


FIG. 3.—DIAGRAM SHOWING DISTRIBUTION OF FORCE, USING 2-BAG MIXER.

A, foreman; B, subforeman; C, finisher; D, 2 laborers striking and tamping; E, 3 laborers placing concrete and assisting in striking; F, mixer tender; G, laborer cleaning subgrade and setting joints; H, mixer engineer; I, fireman, also sprinkles subgrade; J, laborer assisting wheelers and cement handlers; K, 2 laborers handling cement; L, 2 laborers wheeling sand; M, 3 laborers wheeling coarse aggregate; N, 2 laborers loading sand; P, 4 laborers loading coarse aggregate; Q, laborer sprinkling pavement; R, water boy. Total, 2 foremen and 25 laborers. □ indicates wheelbarrow

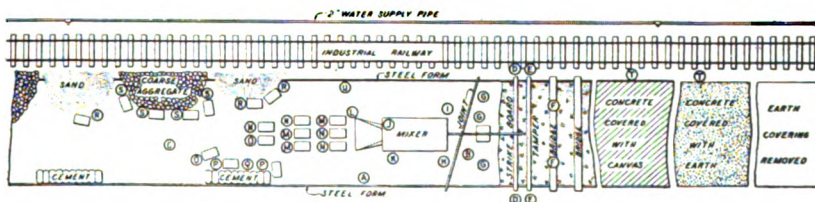


FIG. 4.—DIAGRAM SHOWING DISTRIBUTION OF FORCE, USING 3-BAG MIXER.

A, foreman; B, subforeman on placing concrete; C, subforeman on charging mixer; D, 2 laborers striking; E, 2 laborers tamping; F, 2 finishers; G, 3 laborers placing concrete; H, mixer tender; I, laborer cleaning subgrade and setting joints; J, mixer engineer; K, fireman, also sprinkles subgrade; L, laborer assisting wheelers; M, 3 laborers wheeling sand; N, 4 laborers wheeling coarse aggregate; O, 2 laborers wheeling cement; P, 2 laborers handling cement; Q, laborer opening bags; R, 3 laborers loading sand; S, 4 laborers loading coarse aggregate; T, 2 laborers sprinkling pavement; U, water boy. Total, 3 foremen and 36 laborers. □ indicates wheelbarrow

they have not been durable where the traffic is at all heavy. The uneven manner in which they fail tends to produce excessive wear on portions of the concrete, and renewals should be made promptly as needed.

In constructing bituminous surfaces on concrete it is essential to have the surface of the concrete entirely clean and free from laitance when the bitumen is spread. Generally about one-half gallon of bitumen to the square yard is put on in either one or two applications, by hand or by means of pressure distributors. It is sometimes swept with hand brooms in order to make it adhere better to the pavement. Hot applications have hitherto been almost exclusively used, though there is no apparent reason why materials which could be spread cold might not be employed with equally satisfactory results.

After the bitumen has been spread as described, it is covered with coarse sand, pea gravel, or stone chips, applied at the rate of 1 cu. yd. to from 75 to 100 sq. yds. of surface. The road may be opened to traffic almost immediately after the sand or stone chips are spread.

It is realised that the above discussion of bituminous wearing surfaces falls very far short of furnishing a guide for undertaking work of that kind. The available data upon this subject, however, are not considered sufficient to form a basis for a more comprehensive discussion.

ORDER AND PROGRESS OF THE WORK.

In constructing a concrete pavement it is especially desirable that the work of mixing and placing the concrete be as nearly continuous as practicable after it is once begun. On this account the order and progress of the work should ordinarily be planned with the primary view of keeping the mixer going full time every working day that the weather will permit. This means that ample provision should be made for completing the drainage structures, the grading, and the preparation of the subgrade well ahead of the mixer, as well as for supplying the mixer with all necessary materials.

Rather than construct a concrete culvert sufficiently far in advance for the subgrade to be prepared before the mixer arrives, it may sometimes be economical to leave out a section of the pavement over the culvert. But the extra expense involved in going back and putting in a section of this kind after the work of laying the pavement has progressed a considerable distance ahead is usually considerable and is often underestimated by contractors. This method of doing the work also involves a delay in opening the road, and as a rule is very objectionable on that account.

The work of preparing the subgrade and setting the forms should preferably proceed sufficiently far in advance of the mixer to allow for two or three days' run. The prepared subgrade, if properly drained, dries out much more rapidly after rains than the rough grade, and thus it is possible to resume the placing of concrete much earlier than when the roadbed has not been shaped and rolled. A soaking rain will usually cause the prepared subgrade to heave slightly and make rerolling necessary, but ordinarily this is a very small item.

OPERATING THE CONCRETE MIXER.

In general it is economical to employ a mixer of the street-paving type for mixing and placing the concrete, though in some cases it has proved satisfactory to do the mixing in stationary mixers and haul the concrete out to its place in the road. This latter method is applicable to relatively only a very few sets of conditions.

Organising a force of labourers to operate a paving mixer efficiently requires considerable skill in handling men. The best results are generally obtained when a mixer is fully manned and each labourer is assigned definite work to perform.

HANDLING MATERIALS.

One of the most difficult problems which have to be solved in connection with concrete road construction is that of determining the proper methods to employ in handling and delivering the materials for the concrete. The different kinds of material required must be delivered to the mixer in definite proportions at the same time, and it is evident that the location of the several sources from which the materials are obtained, with respect to each other and to the road, will have a very great influence in determining the most economical transportation methods.

The importance of the water supply is not always appreciated by contractors and engineers, and the provision made for delivering water on the work has sometimes been entirely inadequate. Another frequent error is that of overestimating the amount of water which a chosen stream is capable of supplying. In general, the most practicable method of delivering the water is to pump it through a pipe line laid along the road.

No matter what the arrangements for testing the cement may be, provision should usually be made for storing near the work sufficient cement to keep the mixer going for four or five days, in case that shipments are delayed, as frequently happens.

In general, the most satisfactory method of hauling the materials for the concrete is by means of an industrial railway constructed along one shoulder of the road, though this method is not always the most economical. Teams, traction engines with trailers, and motor trucks with or without trailers have each been frequently used for this purpose, and are no doubt each economically best adapted to certain sets of conditions. But all of these are objectionable from a construction standpoint on account of the damage which they usually do to the subgrade.

MAINTENANCE.

The maintenance of the pavement consists, for the most part, in repairing cup holes, cracks, contraction joints, and perhaps the renewal of an occasional defective area.

Cup holes are spots in the surface of the pavement which break down under traffic and which may result from any one of a number of causes. The most frequent cause for such defects is the presence of sticks, lumps of clay, particles of unsound stone, or other objectionable material in the aggregates. The maintenance of cup holes, cracks, and joints usually consists of filling them with tar and covering the tar with coarse sand, pea gravel, or stone chips. Satisfactory results can be secured by this method only when a crew with proper equipment and materials goes over the road making necessary repairs at least two or three times a year.

Where defects of any considerable size are to be repaired the edges should be chiselled down until they are approximately vertical and not less than about 1 in. deep. The hole should be thoroughly cleaned and painted with tar, after which it should be filled with clean, coarse stone chips thoroughly grouted with tar. The surface of the patch should then be covered with coarse sand, pea gravel, or fine stone chips.

Either refined water-gas or coal-gas tar may be used for making such repairs, and the Office of Public Roads has obtained satisfactory results with both kinds.

When it becomes necessary to renew any portion of the pavement with Portland cement concrete that portion should be entirely closed to traffic, and the concrete should be mixed, placed, and cured in the manner described in the discussion of construction. The edges of the old concrete should be thoroughly cleaned and coated with neat cement mortar before the new concrete is placed.

A properly constructed concrete pavement ought to wear down uniformly and develop few defects. Poorly constructed and poorly maintained contraction joints are probably responsible for more defects of the kind described than can be attributed to any other one cause. For this reason the contraction joints should be given very careful attention at the time of construction.

It has been claimed that the difficulty involved in properly maintaining defects in joints and cracks and the inconvenience attending periodic renewals of the pavement may be largely eliminated by maintaining a bituminous wearing surface over the concrete. Until further improvements are made in this method of treating concrete pavements, however, no specific recommendations can be made.

CONCLUSION.

In concluding this discussion of concrete roads the principal points may be summarised as follows:

(1) The economic efficiency of concrete roads is undetermined at present, but the indications are that this type of construction will prove to be well suited for certain conditions.

(2) The one-course type of concrete pavement is greatly to be preferred to the

two-course type, but there are conditions under which the adoption of the two-course type of construction may be justified.

(3) The proportion of cement to the sand and coarse aggregate combined should not be less than about 1 to 5, and the proportion of sand to coarse aggregate should not be less than $1\frac{1}{2}$ to 3, nor greater than 2 to 3. Ordinarily, when gravel is used as coarse aggregate, the proportions may be made 1 part of cement to $1\frac{1}{2}$ parts of sand to 3 parts of gravel, and when crushed stone is used as coarse aggregate, 1 part of cement to $1\frac{1}{4}$ parts of sand to 3 parts of crushed stone.

(4) All types of contraction joints which have yet been devised require careful and frequent attention in order to prevent rapid deterioration of the pavement in their vicinity. It appears that better results are obtained by spacing the joints at an angle of about 75 deg. to the centre line of the road than when they are placed at an angle of 90 deg.

(5) Thin bituminous wearing surfaces for concrete pavements cannot be economically justified at present. It is possible that through experimental investigations some method of constructing such surfaces to give uniformly satisfactory results may yet be devised. If this is done, the maintenance of concrete pavements and the contraction-joint problem will be greatly simplified.

(6) Intelligent engineering supervision is absolutely essential in concrete pavement construction, because defective materials or workmanship cannot be readily repaired after the pavement is completed, and they are not usually apparent until the pavement has been in use for some time.

THE INSTITUTION OF CIVIL ENGINEERS.

PRESIDENTIAL ADDRESS.

In his opening address as President of the Institution of Civil Engineers, Mr. Alexander Ross said that while each one of us, according to his ability and opportunity, was endeavouring to do his best for the national cause, and for the success of the British arms, we could not ignore the fact that the circumstances had involved great sacrifices among professional engineers, and he feared we must look for a certain amount of distress to follow. Professional engineering work had been adversely affected to an extraordinary degree by the need, which was realised by all, of husbanding the national resources and arresting the prosecution of a large number of those undertakings which in the usual course employed professional engineering skill.

While sharing the view that much expenditure and application of labour which was not directed to military needs should be suspended for the present, we might well doubt whether it was a truly economical course to postpone, even under the present circumstances, the contemplation and professional consideration of engineering works, for which the call might be expected to be acute when the war was over. It appeared to him that many engineers whose age and circumstances prevented them from taking an active part in connection with the war might well be devoting their skill to the deliberate preparation of such projects as he had referred to. The cost necessarily involved would be very small, and there could be little doubt that the country would be in a better position to resume normal activities in the future, if effort which could not be made use of otherwise were applied to the preparation of designs for work which was certain to be undertaken under some pressure when the time came.

Another matter which must exercise our minds in connection with the future of many of the young men who had volunteered to serve the country was what will be the position of those of them who had embarked upon or contemplated an engineering career, which had been completely broken off? It was impossible to create a competent engineer without due training and experience, and we would no doubt have to consider somewhat specially, at a later date, the position of those young men whose preparation for the engineering profession must be, owing to the war, regarded as imperfect, according to the existing rules of the Institution.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

THE SUBSTRUCTURE OF THE LETHBRIDGE VIADUCT, CANADA.

West of Lethbridge City, in Southern Alberta, Canada, there has been completed by the Canadian Pacific Railway an engineering work of great magnitude, and it is said to be the longest and highest bridge in the world. The structure bridges the Belly River, and is 1 mile and 47 ft. long and 314 ft. high. The cost exceeded two million dollars. It required 645 cars to transport the steel actually used in the erection, and 1,000 cars for contractors' plant, timber ties, guard rail, cement, and other materials. Dry excavation totalled 18,110 cu. yds., and wet excavation 4,870 cu. yds. There were 17,090 yds. of concrete, 1,676 concrete piles, 339 cu. yds. of riprap, and 12,200 tons of steel used in construction.

The general formation of the river bed is gravel overlying hard shale, with an average depth of 10 ft. of water. The river piers were carried down to good foundation on the hard shale. For about 20 ft. below the bed of the river there is gravel; then 20 ft. of coal shale; below that 2 ft. of blue clay; then a hard shale for a distance of 16 to 18 ft., below which is hardpan and sandstone.

The excavation for these piers, before caissons were placed, was made with clam-shell buckets. The caissons were made of 10 in. by 10 in. timber securely braced, floated into their places, and sunk in proper position. Sheet piling, made of three pieces 2-in. by 10-in. planks nailed together to form a tongue and groove, was driven outside the caissons, and a second row of sheet piling about 3 ft. outside the first. The intervening space was filled with clay puddle. The caissons were pumped out with three centrifugal pumps, when the excavation was completed with pick and shovel, and the material handled by bucket and derrick. When the excavation had reached hard shale, drillings were made to a depth of at least 20 ft., to make sure that there was no change of formation.

In three of the piers it was found that the pumps could not hold the water down, and it was therefore necessary to have the bottoms carefully cleaned and prepared by divers before the concrete was deposited. A layer of concrete 4 ft. in depth was then placed in the bottom, under water, over the full area of the caisson and allowed to set, after which the caisson was pumped out and the balance of the pier finished in the dry.

Much difficulty was encountered in driving the concrete piles on the steep hill-sides, and it was found necessary to make considerable alterations in the pile-drivers and cribbing on which to block them. It was necessary to carry down all foundations to such a depth that they would be below the natural slope of the ground if the steep banks continued to erode.

The toe of the west bank along the river edge was protected from erosion by constructing a reinforced concrete retaining wall for a considerable distance up and down the stream from the site of the bridge, the up-stream end terminating in an exposed seam of coal shale. Along the flat on the east side of the river clusters of piles were driven up-stream from the towers, to catch and deflect any drift-wood that might come down in flood. In addition to these piles, heavy deflecting booms have been located in the river opposite the towers for the same purpose.

Gravel taken from the bed of the river was used throughout and was found to be such that no screening was required. It is interesting to note here also that sufficient coal was taken from the excavations to run the engines supplying power for the work of excavation.

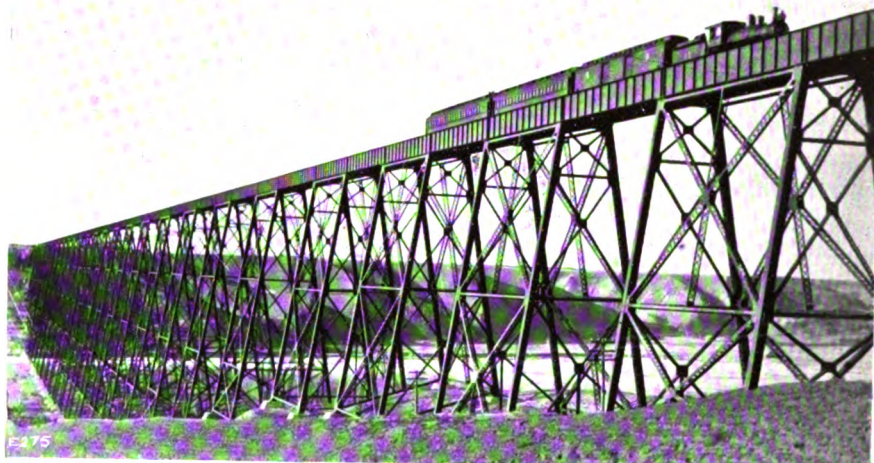


Fig. 1. General View of Viaduct.

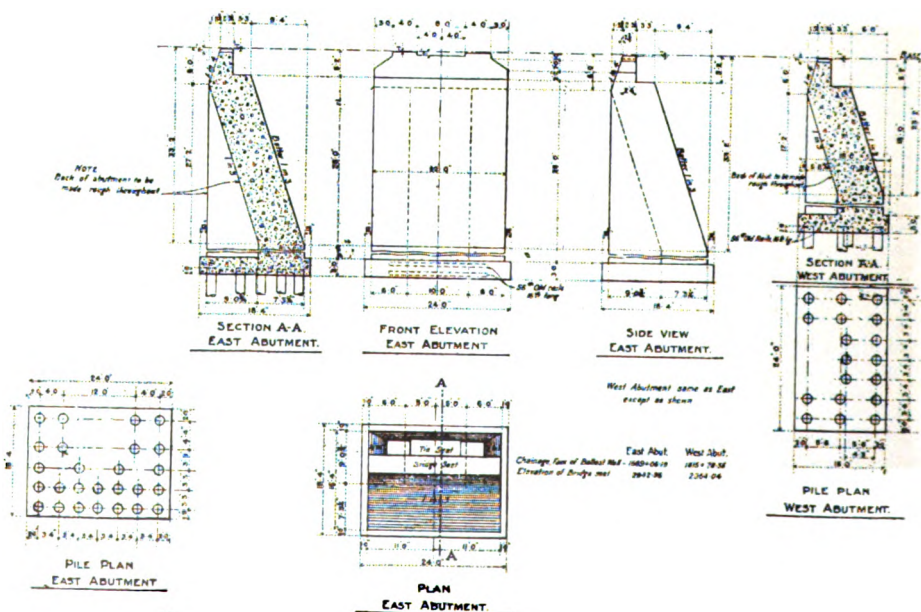


Fig. 2. Showing Concrete Abutments.

THE LETHBRIDGE VIADUCT SUBSTRUCTURE.

The specifications for concrete piling provided that piles should be of approved form, and vary in size according to length, all piles to be placed by first driving a core, no driving of the concrete piles themselves being allowed. The concrete forming the piles consisted of one part cement, three of sand, and six of broken stone or gravel not exceeding $\frac{3}{4}$ in. in size. The diameters of the piling for a 30-ft. length was 20 in. at the butt, and for 35 ft. to 40 ft. long, 18 in. at the butt, with the minimum diameter of

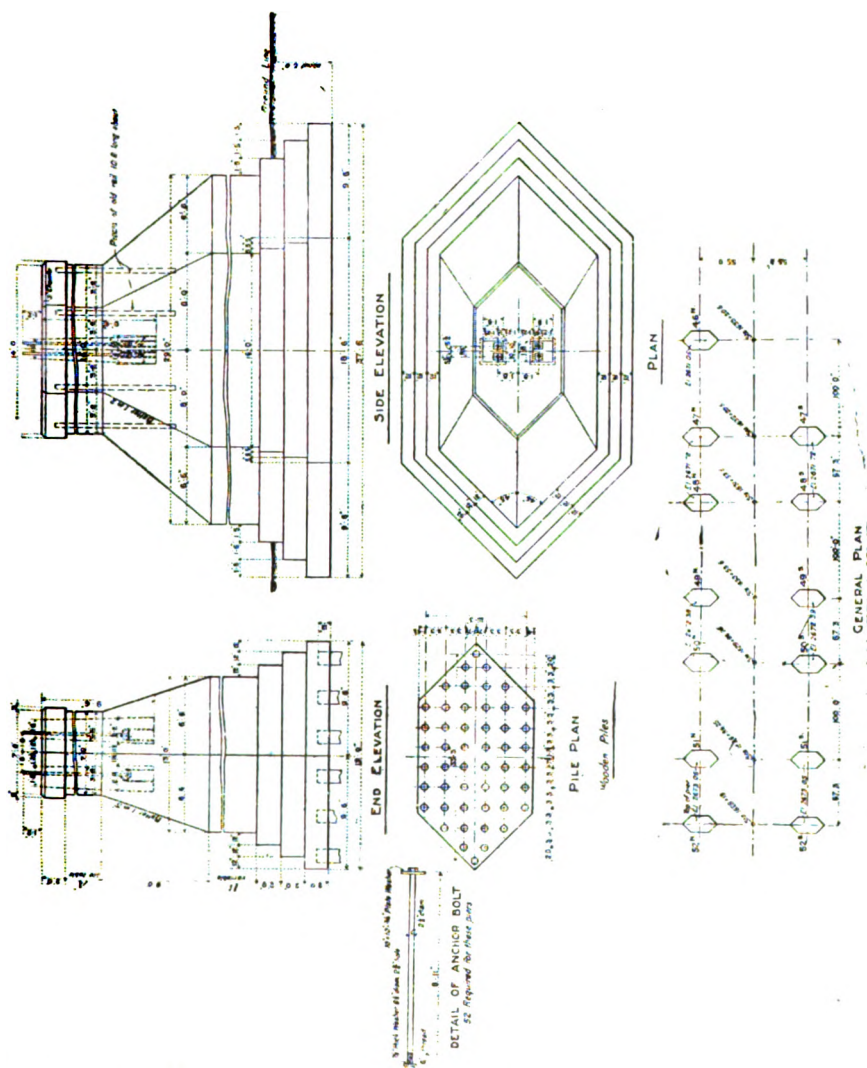


FIG. 3. Showing Concrete Piers in River. THE LETHBRIDGE VIADUCT SUBSTRUCTURE

6 in. at the point. It was not considered advisable to use wooden piles, as the foundations would be alternately wet and dry. The type used was the Raymond tapered pattern, which consists of sheet iron casing, inside of which is a collapsible core, the latter being withdrawn when the pile is driven, and the casing then filled with concrete.

After the piers were completed anchor bolts were set in place accurately as to centres and height, and grouted in for about half the length of the bolts, the balance of the bolt being left free so that it could be sprung, if necessary, when erecting the steel-

work. The tops of these holes projected far enough beyond the edge of the bed plates to enable them to be completely filled with grout on completion of the structure.

The bridge was designed in the office of the Engineer of Bridges of the Canadian Pacific Railway Co., at Montreal; the sub-structure was laid out and built under the

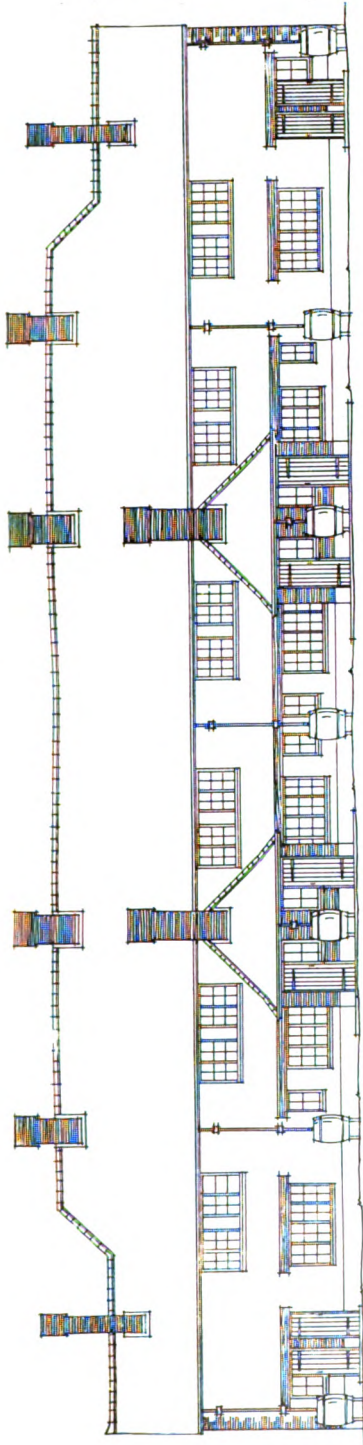


Fig. 4. Showing Concrete Footings.
THE LETHBRIDGE VIADUCT SUBSTRUCTURE.

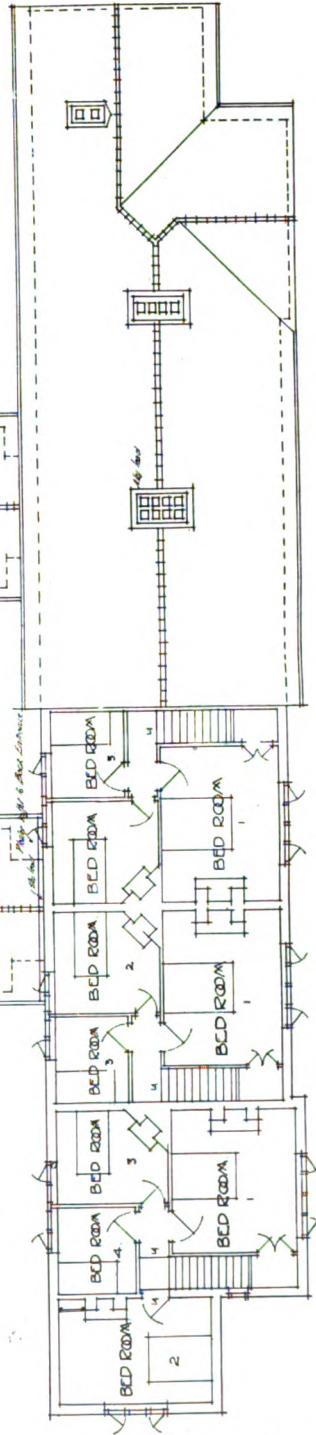
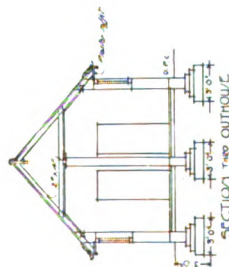
direction of the Assistant Chief Engineer, Mr. J. E. Schwitzer, of Winnipeg, Man.; and the erection of the steelwork was carried out by the Bridge Engineer, Mr. J. Rankin. Mr. C. C. Schneider, of Philadelphia, was consulting engineer. The Canadian Bridge Co., of Walkerville, Ontario, had the erection of the steelwork, Mr. A. L. Colby being manager of construction.

SOME CONCRETE COTTAGES AT NORWICH.

THE accompanying illustrations show some cottages built at Whittingham Sewage Farm, Norwich, of concrete. The blocks were moulded on the site on a Winget machine, the gravel being raised on the farm. The cost of the cottages was about the same as if they had been built of local bricks—perhaps somewhat less. Approximately, the total cost, including the blocks, was about £1,700.



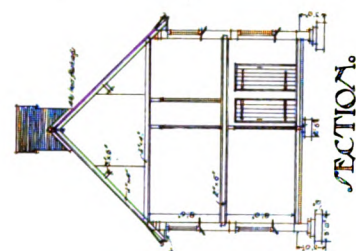
BACK ELEVATION.



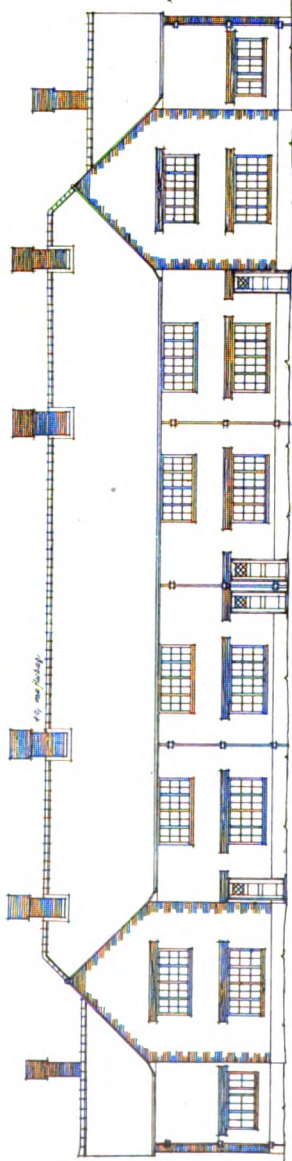
ROOF PLAN.

1ST FLOOR PLAN.

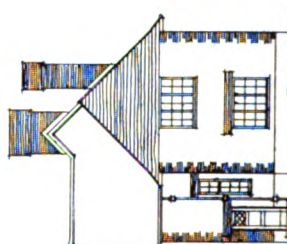
CONCRETE COTTAGES, NORWICH.



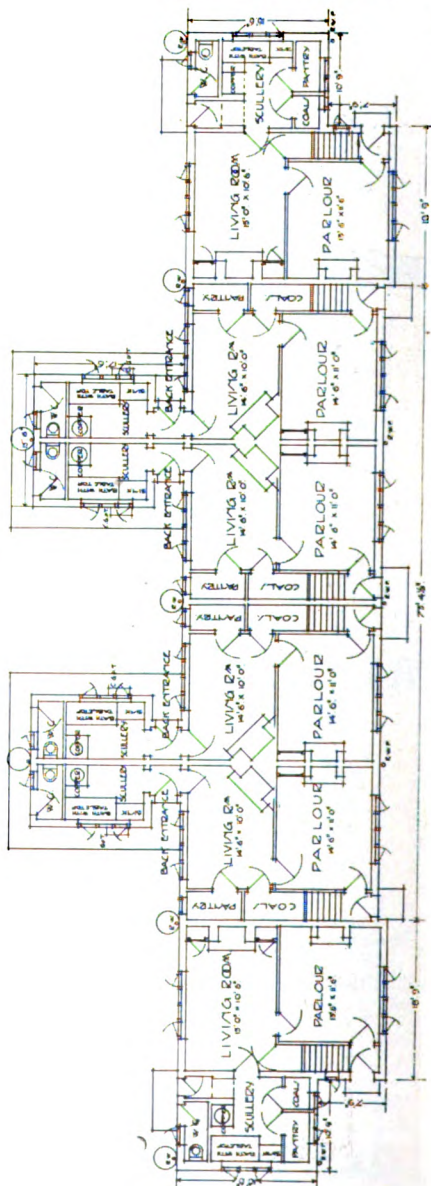
SECTION.



FRONT ELEVATION.



SIDE ELEVATION.



GROUND PLAN.

CONCRETE COTTAGES, NORWICH.

All the walls are hollow. There has never been a vestige of damp in any of the cottages since they were built. They were completed about two years ago.

The cottages were designed by Mr. Arthur E. Collins, M.Inst.C.E., City Engineer of Norwich, to whom we are indebted for our particulars and illustrations.



GENERAL VIEW OF **CONCRETE COTTAGES, NORWICH.**

NEW BOOKS AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

A Text Book on Practical Mathematics for Advanced Technical Students. By H. Leslie Mann, B.Sc., A.R.C.Sc.

London: Longmans, Green & Co., 39 Paternoster Row, London, E.C. 487 pp. 8½" x 5½". Price 7/6 net.

Principal Contents.—Binomial Theorem, Approximations—Solution of Triangles—Complex Quantities and the "Imaginary" ($\sqrt{-1}$)—De Moivre's Theorem—Graphical Representation of a Complex Quantity—Hyperbolic Functions—Co-ordinate Geometry—Differentiation, Maxima and Minima—Curvature and Radius of Curvature—Point of Inflection—Integration—The Centroid—Moment of Inertia and the Principal Axes of Inertia—Momental Eclipse—Bending Moment and Shearing Force—Deflection of Beams—Distribution of Shear Stress over a Section—Twisting Moment—Motion of a Projectile—Theory of Struts—Euler's Formula—Gordon—Rankine Formula—Vector Analysis—The Vector Product—Answers—Index.

It is a real pleasure to handle and review such a book as this. The type is clear and the general get-up of the book is quite good.

The contents of the book are well thought out and are obviously the work of a trained teacher. Unfortunately many books are written by those who, having the knowledge themselves, yet do not possess the faculty or the gift of imparting that knowledge to others.

So as not to make the work too cumbersome for a text book a preliminary knowledge of the fundamental principles of Algebra, Trigonometry, and Mensuration, and the use of Logarithms and squared paper has been assumed, this being well within the scope of the elementary student. This book is meant to cover a two or three years' course, so, obviously, its contents are not to be devoured at a single sitting, interesting as they are.

The general equation—

$$\frac{1}{r} = \frac{d^2y}{dx^2} \left[1 + \left(\frac{dy}{dx} \right)^2 \right]^{\frac{3}{2}}$$

which approximates to

$$\frac{1}{r} = \frac{d^2y}{dx^2}$$

as the curve becomes flatter and flatter is quite fundamental in the investigation of the flexure of beams and pillars.

The author decides wisely in giving a diagram which shows positive curvature.

Some authors give a diagram showing negative curvature and then proceed to apply the rule to beams having positive curvature without the slightest explanation or apology for their *volte face*.

In many treatises the derivation of the equations for curvature are slurred over. We have examined the point in about twenty different text books and scarcely one of them gives an exposition which is worthy of the importance of this equation to the structural engineer.

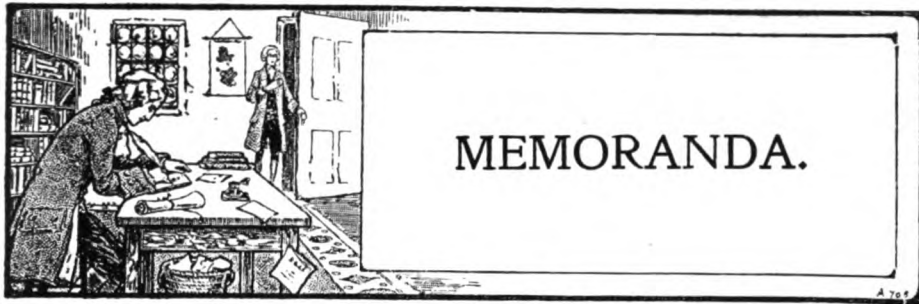
In most of the cases intermediate steps of the proof are omitted, and it is only the more highly trained engineers who would be able to supply the missing details in the proof of an equation which lies right at the foundation of the Theory of Structures. Even the present author's proof is much too laconic for the majority of engineers; and if succeeding editions were to contain a proof showing all the algebraic and trigonometrical substitutions on the way to the statement of the fundamental theorem that $\frac{1}{r}$ approximates to $\frac{d^2y}{dx^2}$ in the

case of flat curves, he would have deserved the thanks of engineers to a greater extent than he realises, even though in the preface he recognises that a great many of the difficulties which arise out of calculus work are entirely due to a weakness in the knowledge of Algebra and Trigonometry.

The sections dealing with the deflection of beams are quite good and clearly illustrated.

The book is well arranged and contains sections dealing with Electrical Engineering and matters other than our special field of interest—Concrete and Constructional Engineering.

The parts having special applicability to concrete and other structural work have been set out in our introduction.



Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The Concrete Institute.—The winter session of the Institute was opened last month, when Prof. Henry Adams, M.Inst.C.E., delivered his second Presidential address, of which a report will be found on another page.

There will be ordinary general meetings of the Institute at Denison House, Vauxhall Bridge Road, S.W., at 5.30 p.m., as follows:—January 19th, 1916, February 16th, March 15th, April 19th, and May 17th. A Paper will be read at each meeting, the subject matter of which will be announced at a later date.

Midland Institute, Birmingham.—A lecture was delivered last month by Mr. Harry Jackson, A.M.Inst.C.E., before the Midland Institute, on "Reinforced Concrete." The lecture dealt in a general way with the different stresses which are taken up by various parts of reinforced concrete structures, each particular kind of stress being illustrated by a number of examples from the lecturer's designs.

Various methods of carrying out actual constructions from design were dealt with, fixing the centering, steel and concrete, and reference was made to several special difficulties which have had to be overcome in the actual execution of designs.

Towards the end of the lecture a little consideration was given to what concrete actually is, and some theoretical reasons were adduced for the very generally accepted proportions of 6 : 1 for reinforced concrete work.

These points were all illustrated by lantern slides.

Police Warning—Hostile Air Raids.—We have been requested to publish the following notice:—

In view of the possibility of further *Attacks by Hostile Aircraft*, the Commissioner of Police deems it advisable to call attention to the public warning published on June 28th, recommending residents to remain under cover and advising them, for dealing with incendiary fires, to keep a supply of water and sand readily available. The opportunity was taken of suggesting that chemical liquid fire extinguishers should not be purchased without a written guarantee that they comply with the specification of the Board of Trade, Office of Works, Metropolitan Police, or some approved Fire Prevention Committee.

Many dry powder fire extinguishers are much advertised. The Commissioner warns the public, as the result of experiments made, at his instance, by a competent Committee of Experts, that no reliance can be placed upon such appliances for effectively controlling fires such as are likely to be caused by bombs, explosive or incendiary. The Commissioner is advised as the result of these experiments that the provision, and prompt and intelligent use of water, or of sand, or of both, in dealing with such outbreaks of fire is the best, simplest, and most economical safeguard.

(Signed) E. R. HENRY,

The Commissioner of Police of the Metropolis.

Metropolitan Police Office,
New Scotland Yard.
September 18th, 1915.

Houses for the Working Classes.—Discussing the work done during the year by the Edinburgh Dean of Guild Court, Mr. J. Macintyre (Lord Dean of Guild) referred to the attempts made to reduce the cost of building for the working classes. He pointed out that for many commercial and public buildings steel and concrete construction was becoming so very largely used that its possibilities of effect upon the economies of building construction in the future could not be foreseen and might be very great.—Plans have been before the Tipton Urban District Council for the erection of workmen's houses in reinforced concrete, and the Council have agreed to allow two houses to be built as a sample. They will be built of slabs of reinforced concrete already prepared, and if not approved they will be demolished.—Sixty-nine houses have been erected in the Renfrew Garden Suburb from the designs of Mr. W. Inglis, architect, of Glasgow. The external walls are built of concrete blocks, with a 2-in. cavity and a total thickness of 9 in. Externally they are treated with rough-cast. The interior partitions are formed with coke breeze concrete slabs. The accommodation given in the houses varies from two to five compartments, and each house has a bathroom, scullery and pantry. The living rooms in the smallest houses are 12 ft. square. The rents range from £15 to £27 per year.

Lining the Shafts of Mines—A Downward System.—A paper by Mr. Marcel Gillieaux, read before the Mining Institute of Scotland, described a method of lining the shafts of mines from the top downwards by means of concrete blocks of special shape.

He remarked that if it were feasible to build a concrete lining downwards—that is, from the top to the bottom, following step by step the deepening of the shaft—it would be possible to dispense with all temporary supports, the defective junctions between the sections in which it must be constructed would disappear, and the workmen, being in security, would work better and faster. So far only incomplete and unsatisfactory solutions of the problem have been proposed, more complicated than the old methods while possessing all their defects.

USE OF Z-CONCRETE BLOCKS.

Concreting downwards can, however, be carried out by using blocks of such a shape that they are capable of self suspension, and to that end the author has devised a form of construction in which the segments are of Z-section. Each section is hooked on to the next higher section, and thus, on account of the weight of the blocks, a closer joint is made than is possible in the ordinary way. The method dispenses with all temporary shoring, expensive bolts and ties, and defective junctions, and realises the idea of a homogeneous wall. A strong crown-ring is let into the standing rock at the head of the lining, and when the sinking has progressed sufficiently, a series of segments of special shape is suspended from it to form the first ring of the tubing. Additional rings are then hooked on to those already in position, the rib of the segment that forms the upper hook being directed outwards, while that which forms the lower hook is directed inwards. When several rings have been put in position, the annular space between the lining and the shaft wall is plugged, and a semi-fluid mixture of cement and sand is injected through holes provided for the purpose. When the cement has set it binds the lining firmly to the external rock, and additional blocks can be hung on below without fear of straining the upper portion. The blocks are reinforced with S-iron bars forming a double hook extending from the top to the bottom rib, with auxiliary simple hooks arranged symmetrically to the first-named in the nose pieces and throat sections.

The cost of lining in this way must differ according to circumstances, but as an example the author took a circular shaft 18 ft. in inside diameter, sunk through clay or schist, with blocks 7½ in. thick. The blocks would measure 2 ft. 3¼ in. on the inside arc length and 2½ ft. in height, each ring containing 24 blocks. Such blocks, similar to those used in a trial by the author, cost about 4s. each, or less in larger quantities. For a lineal yard of lining the blocks will cost about £5 15s. The cost of setting depends on many circumstances, but from the author's experiments it may be estimated at 6d. per block, or about 15s. per lineal yard. Filling the space between the blockwall and the side of the shaft is the most difficult item to calculate, but in the kind of soil mentioned it will certainly be less than £3 and perhaps not more than £2 to £2 10s. per current yard. The total cost of the lining will be less than £9 10s. per lineal yard.—*Times* (Engineering Supplement).

Oil Mixed Concrete Tests.—In our October number we published an article on oil-mixed concrete by Mr. Logan Waller Page, but owing to lack of space we were compelled to hold over the following interesting particulars regarding tests carried out by the Office of Public Roads in Washington. These particulars were as follows :—

A number of firms, corporations, and individuals applied to the Office of Public Roads, Washington, for information in regard to using oil-cement concrete in various structures. In these cases the office supplied the oil specifications and directions for mixing and applying the materials, but no supervision. Later, inquiries were made relative to the success met with where the specifications had been followed. Many of these inquiries were not answered, but of the twenty-nine replies received from persons who had used the oil-cement concrete, only three were wholly unfavourable, while one was partly so. Of the three unfavourable replies, one referred to the use of the material for paving blocks, and another to its use in the construction of tanks for holding acids.

A summary of these twenty-nine replies is given in the following table. As considerable difficulty was encountered in securing oils that met the specifications, substitute oils were used in some instances.

TABLE 1.—RESULTS OBTAINED IN THE USE OF OIL CONCRETE AS A WATERPROOFING MATERIAL IN ACTUAL SERVICE.

Results Favourable.

Reply No.	Nature of Work.	Per Cent. Oil Used.	Proportions and Consistency.	Character of Workmanship.	Character of Waterproofing Required and Results Obtained.
1C	Lemon washing tank : floor for toilet ; plaster partitions on metal lath	5	—	Expert ..	Results satisfactory in every way
2C	Retaining wall ..	10	1 : 3 : 5 (wet)	Expert ..	Waterproof interior against damp earth. Results generally satisfactory
3A	1½-in. finish on exposed floor over water tank	1 gallon to bag cement	Bag cement to 4 barrows sand	Expert ..	To waterproof floor against percolation of rain water. Results good. Oil cement laid twice as fast as that without oil
4A	Swimming pool, 20-in. wall, originally waterproofed with "Ceresit." Unsatisfactory (5 ft. water in tank)	1 gallon to bag cement	Bag cement to 4 barrows sand	Expert ..	Scratch coat ½ in. thick plastered on inside of tank. Tile laid on this. Over 2 years old and has never leaked. Tile adhered well
5B	Roof of tool house and silo	10	1 : 6	Not expert	Plastered 1 in. on metal lath. Results very good
6B	Cellar of dwelling house	¾ gallon to bag	1 : 2 : 2	Not expert	1 ft. water in cellar before waterproofing. Absolutely watertight after using oil concrete
7A	Cellar walls and floor 21 by 43 by 5 ft. ; walls 18 in.	10	1 : 2 : 4	Expert ..	To waterproof against supersaturated earth filling. Excellent results. Feels assured as to value of oil for this class of work
8A	Rain-water cistern 18 in. under ground ; 15 by 15 by 10 ft. ; 14-in. walls	—	1 : 3 : 5	—	Probably tight. No water comes into cellar 6 ft. distant
9A	Water tank — inside walls lined with mortar containing hydrated lime and "Aquabar"	10	1 : 3 : 3	Expert ..	Oil appears to be beneficial

TABLE 1.—RESULTS OBTAINED IN THE USE OF OIL CONCRETE AS A WATERPROOFING MATERIAL IN ACTUAL SERVICE—Continued.

Results Favourable. - Continued.

Re- ply No	Nature of Work.	Per Cent. Oil Used.	Proportions and Consistency.	Character of Work- manship.	Character of Water- proofing Required and Results Obtained.
10A	Water tanks of various sizes and floors	5	1 : 2	Expert ..	Oil mortar used to plaster inside of tanks. Results satisfactory. Floors satisfactory. Dry up quickly after flushing. Mortar "works" much better than plain
11A	Floor of machine shop 12 by 15 ft.; 2-in. top	5	1 : 2	Expert ..	Construction recent, but apparently satisfactory
12B	Floor in corn bin; and a poultry house com- plete (floor, walls, roof)	5	1 : 3 20 per cent. hyd. lime	Not expert	Roof tight except at one place where crack formed. Results good
13B	Walls and floor of hog house	5	1 : 2	Not expert	Recommends use of oil Damp-proofing desired. Great success
14B	Foundations (8 in. thick) and cistern (12 ft. deep)	5	1 : 3 : 4 (wet)	Moderately expert	Walls are perfectly damp- proof
15B	Floor of grain bins ..	15	1 : 1 (wet) ..	Not expert	Is perfectly hard and waterproof
16B	Paving block 12 by 14 by 5 ft.; oil mixture used as facing	2	1 : 3 (very wet)	Not expert	Makes hard, very smooth surface, impervious to moisture because non- porous
17A	Concrete silos.. ..	5.	1 : 2½ : 4½ 1 : 2 : 4 (wet)	Expert ..	Results generally satis- factory
18B	Concrete arch; facing of soffit	10	1 : 2	Expert ..	Waterproofed concrete in excellent condition. Non- waterproofed concrete badly disintegrated
19B	Plaster coat to inside of cistern ½ in.	10	1 : 4 (wet) ..	Expert ..	Waterproofing seemingly perfect. No known deterioration
20B	Renovating leaky cis- tern; 7/8-in. coat fol- lowed by brush coat	20	Neat ..	Not expert	Results very satisfactory. Absolutely waterproof
21B	Cellar wall, 4-in. plaster coat	8	2 : 3 (wet) ..	Not expert	Wall so far waterproof
22B	Basement walls of a building subject to action of ground water	2	1 : 1 : 3	—	Absolutely waterproof and damp-proof. Stone wall above continually ad- mits water
23B	Tank for wood pulp ..	(¹)	(¹)	¹)	Results entirely satis- factory
24A	Lining a cellar wall, subjected to 4-ft. head of water; wall 6 in thick	10	1 : 2 : 4 (wet)	Expert ..	Only trouble slight sweat- ing through joints
25A	Vault walls, basement floor, freight elevator pit	6	1 : 3 : 4	Moderately expert	Wall sustains weight of street, is waterproof and very satisfactory. No other method of damp- proofing so thorough and satisfactory; none so inexpensive

Results Unfavourable.

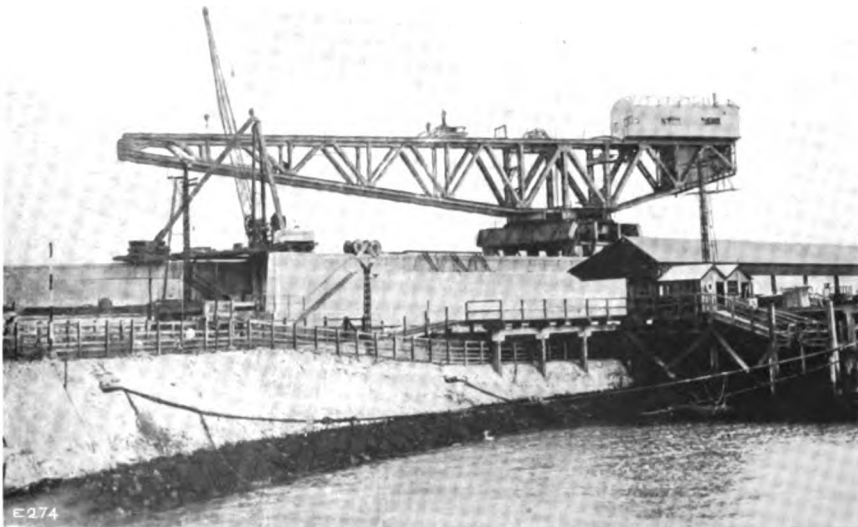
1B	Cement blocks for track paving	10	1 : 2 : 4 (wet)	—	Not successful. Blocks never used. Oil was added for toughness
2B	Tank to hold 1 per cent. soluble sulphuric acid at 150° F.	(²)	(²)	—	Not successful. Acid dis- solved concrete
3B	Floor of cellar 2-in. top	6	1 : 3 (very wet)	Expert ..	Has as much water as before coating was applied
4	Cellar ½ in thick ..	(¹)	(¹)	Not expert	Keeps about one-half volume of water out of cellar

¹ Used according to specifications.² No data kept.

Concrete Vats for Olive Oil.—Whilst concrete is unattacked by tar or petroleum, fatty oils soon attack it, owing to the liberation of fatty acids as the oil becomes rancid. An article in *Il Cemento* deals with the means of making oil vats of concrete or reinforced concrete sufficiently resistant. A rich, well graded mixture must be used, the surface of which may be treated with silicates or fluosilicates. Also, as it is the lime of the cement which reacts with the acids, the addition of 20 per cent. of finely ground pozzuolana is of great advantage. Experiments on this point have been made by Professor Canevazzi, who finds that such an addition confines the action of the acid to an insignificant surface layer. Many of the large underground reservoirs for olive oil in Italy have, however, been lined with slabs of Marseilles red ware cemented together by clay of the same kind as that used for the slabs. These have given good results.

Forcing Frames in Reinforced Concrete.—The use of reinforced concrete for the construction of gardeners' forcing frames is described in *I Materiali da Construzione* for October. Each compartment is conveniently made 2 metres long and 1 metre wide, the walls being 55 cm. high at the back and 42 cm. in front, with a thickness of rather less than 5 cm. The slabs are reinforced with wire netting, and are prepared beforehand. They are then erected and fixed in place by T-irons, protected by painting. Each T-iron is set in a concrete block to prevent sinking. The glazed covers rest on small projecting blocks.

Fishguard Harbour Works.—Our illustration shows a portion of these works in progress, and another crane used for the block laying, which is used in conjunction with the one illustrated in our August issue.



Four and a Half Miles of Concrete Fence Flank Transit Route.—There has recently been completed on the Sea Beach Line rapid transit improvement in Brooklyn, New York, about four and a half miles of reinforced concrete fence built in place.

This fence is 4 ft. 6 in. high, supported on reinforced concrete posts 7 in. by 7 in. by 4 ft. 10 in. high, with alternate 7-in. by 9-in. posts for expansion, spaced 8 ft. 4 in. centre to centre. The latter posts were built with 1 $\frac{3}{4}$ -in. by 3-in. grooves, dovetailed to allow the expansion of the panels. The posts are reinforced by two $\frac{3}{4}$ -in. twisted steel rods and four $\frac{1}{4}$ -in. rods, one at each corner, and the panels by six $\frac{1}{4}$ -in. horizontal rods

running through the posts and eleven $\frac{1}{4}$ -in. vertical rods to each panel. All the rods are tied together by Curry tiers, so as to keep them in position and at proper spacing.

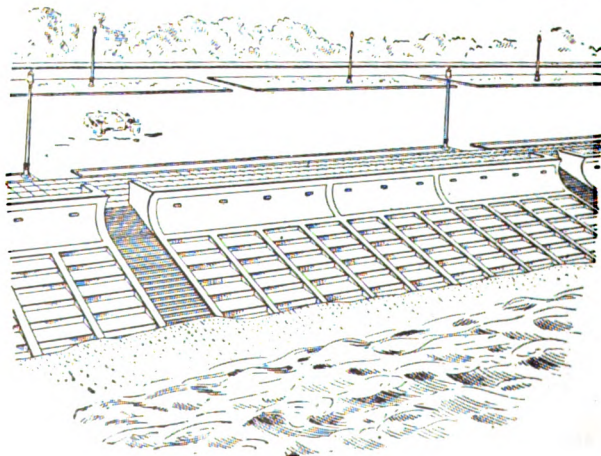
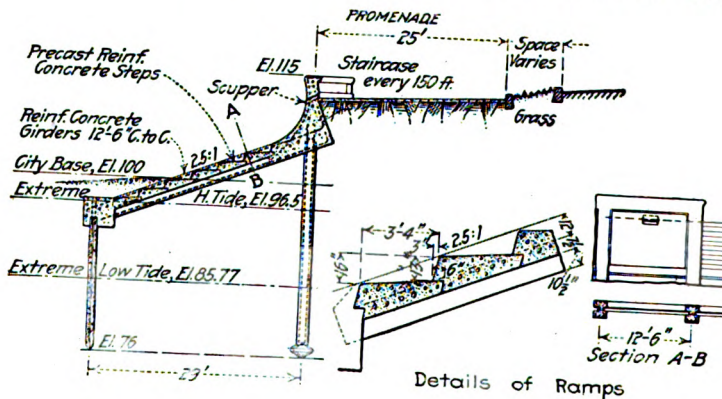
The posts were constructed first. The $\frac{3}{4}$ -in. anchor rods extending into the retaining wall 12 in. are set in grout and are 5 ft. 6 in. long. Several days after the posts were cast the panels were built by the use of forms of good spruce lumber covered on the face with sheets of galvanised iron. The frames were made very rigid and thoroughly braced so that they could be used several times.

The concrete consisted of one part Portland cement, two parts sand, and four parts grit (small pebbles), and was mixed by hand, as only a few yards were required to run 200 to 300 lin. ft. of fence in a day.—*Engineering News*.

Shanghai Electricity Works.—In connection with the improvements and extensions to the electricity works at Shanghai, the new pump house is built entirely of reinforced concrete, the bottom of the building being at a level about 20 ft. below the basement of the turbine room. The building contains three circulating pumps of the vertical shaft centrifugal type, which are driven by a vertical motor at the top of the pump house.

Reinforced Concrete for Bank Safes—A new branch of the Trustee Savings Bank, at Selby, Yorks, was opened early last month, and one of the features is a small new safe constructed of reinforced concrete and fitted with slate shelving and steel door.

San Francisco Shore Protection.—The design for the protection of the ocean beach alongside the new esplanade in San Francisco has just been completed. The esplanade is a new boulevard on the Pacific coast side of San Francisco. In its prepara-



tion it is necessary to protect the beach against the inroads of the sea, and the wall and paving shown in the accompanying drawings are intended for that purpose.

The beach protection consists of a series of steps six in number, which will be erected from beneath the surface of the sands to a position well above the influence of ordinary breakers. Backing the steps is a sea-wall of reinforced concrete with a front moulded to a curve that will throw the breakers back upon themselves and thus counteract their destructive force. Every 150 ft. along the breakwater a concrete stairway will lead from the beach to the scenic boulevard, or esplanade, which will parallel the steps on the east. In front of the steps reinforced concrete interlocking sheetpiles will be driven 15 ft. below mean tide as a protection against the underscour of the seas, and as a foundation for the reinforced concrete sea-wall pedestal piles with mushroom bottoms will be driven at intervals of 10 ft.

Between the coping and the Great Highway, or esplanade proper, will be a promenade 25 ft. in width, of artificial stone. This will be built on a slight slope draining toward the parapet wall, through which will be scupper holes to release any water from wave crests which may top the parapet.

The price estimated for the construction of the first section, to be constructed along the beach for a distance of 650 ft. southward from a point near the terminus of Cabrillo Street, is \$48,500. The design and construction are under the direction of M. M. O'Shaughnessy, City Engineer of San Francisco.—*Engineering News*.

How to Build an Engine Foundation.—The concrete should be mixed in the proportion of 1 part Portland cement, 3 parts clean, well-graded sand, and 5 parts crushed stone or gravel. All parts should be determined by volume and the use of accurate measuring boxes should be rigidly observed. The sand should all pass $\frac{1}{4}$ -in. mesh sieve, and the crushed stone or gravel should pass readily through a $1\frac{1}{2}$ -in. sieve.

After the exact location of the centre line of the foundation has been carefully established, a pit 2 ft. to 4 ft. deep should be excavated, the length and width being the exact size of the footing. Deposit the mushy wet concrete to the depth determined on the plan. In order to thoroughly key the engine foundation to the footing, embed 3- or 4-inch stones in the portion of the footing under the engine base should be carefully set over the footing.

A box form 8 in. larger in length and width than the engine base should be carefully set over the footing. The inside of the forms should be thoroughly oiled to prevent the concrete from adhering to same. It is essential that the anchor bolts for the engine be carefully spaced and so placed as to take care of any small variations in position. Use a templet for this purpose, and supply for each bolt greased gas-pipe of twice the diameter of the bolts, the pipes to be removed before the engine is set. The purpose of the pipes is to provide for such slight adjustment of bolts as may be required. The anchor bolts should be embedded in the concrete at least 18 in., and supplied with cast-iron washers at the lower ends.

After the templet has been accurately set over the forms and the bolts so arranged that the tops are a proper elevation, the concrete is carefully deposited and spaded in the forms. Turn the gas-pipes from time to time, thus preventing them from sticking to the concrete. The concrete along the forms should be carefully spaded to prevent the formation of air-bubbles or pockets.

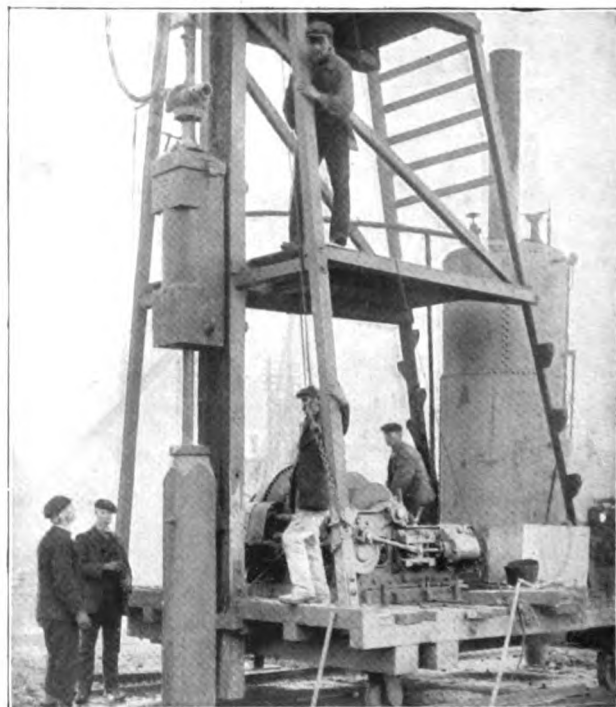
Damp burlap should be placed over the form after the concrete is placed. This will ensure the normal setting of the material. After twenty-four hours remove the form. The engine may be set and the bolts adjusted after forty-eight hours. Before the engine is set remove the gas-pipes referred to above, and when the engine is finally placed fill the space around the bolts with 1:1 mortar.

Do not use the engine until the base is at least two weeks old.—*Concrete Age*.

Grape-Vine Posts of Concrete.—A grape trellis was erected more than a year ago on a farm near Chalfonte, Pa. Realising that construction designed for this purpose is always subjected to strain, the owner decided to adopt the most substantial type. This, in the long run, also meant the cheapest type. For these reasons concrete posts were selected. Concrete posts have such rigidity and strength that they can be planted farther apart than is customary with wooden posts, thus requiring fewer in number. They keep in perfect alignment and there is no decay at any point, whereas wooden posts soon rot at ground level and eventually decay throughout.

The BRITISH STEEL PILING CO

A · P L A N T · F O R · E V E R Y · P I L E



A · P I L E · F O R · E V E R Y · P U R P O S E ·

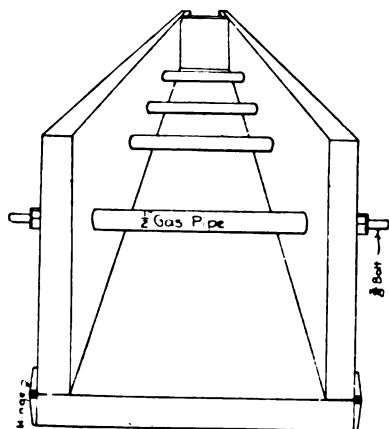
Photograph shows one of our complete PILE-DRIVING PLANTS, on SWIVELLING wheels, 30-cwt. single-acting STEAM HAMMER, double-barrelled STEAM WINCH, and suitable boiler. The piston of hammer passes through base of cylinder (which strikes the blows)—thus whole WEIGHT of hammer is CONSTANTLY ON THE PILE

WEIGHTS OF PILING ON SALE OR HIRE:—

"SIMPLEX"	-	22 to 27 lbs. per sq. ft.
"UNIVERSAL JOIST"	-	from 43 lbs. "

DOCK HOUSE, Billiter Street, LONDON, E.C.

The posts were made of concrete consisting of 1 part Portland cement, $1\frac{1}{2}$ parts sand, and 3 parts small stone. They are reinforced by placing in the corners of each post, about 1 in. from the surface, a $\frac{3}{8}$ -in. square twisted rod. Three-eighths-inch round rods could be used in place of square rods. The posts are 10 ft. long, of which



FORM FOR CASTING CONCRETE TRELLIS POSTS.

4 ft. is beneath the ground and 6 ft. above the ground. They are 6 in. square at the lower end, tapering to 4 in. square at the top. Through each post five holes are provided to receive the wires, upon which the vines are trained. At the end post the wires are fastened to eye bolts, $\frac{3}{8}$ in. in diameter, and about 12 in. long. These bolts extend through the end post and are threaded with a nut for tightening the wire. The bolts and fastenings are clearly shown in the illustration.

In the construction of the Chalfonte trellis the slanting brace for the end post was cast in place. A notch was left in the post and the form for the brace was set up. The reinforcement of the brace projected beyond the end of the form and into the notch, the end of the form merely fitting up against the post. At the ground end of the brace an excavation was made. When the concrete was placed this excavation was also filled, thus forming a bulb or enlarged end, which answered the purpose of an anchor. The brace

form was then filled with concrete, which was worked into the notch, entirely surrounding the ends of the reinforcing rods, the latter being curved at the ends to firmly anchor them. It would be feasible, however, to precast the brace, merely providing for it a notch in the post. The notch could be easily formed by nailing a triangular block to the side of the form. The post end of the brace should be set in cement mortar.

The form for casting the posts is very simple. It consists of three boards, which form a trough the exact size of the finished post. The side boards are attached to the bottom piece by hinges so that they can be readily swung down when the post is removed from the mould. The wire holes are established by placing at the proper points short pieces of $\frac{1}{2}$ -in. gas-pipe cut so as to fit between the side boards of the form. Through these short lengths of pipe, which are left in the concrete, are placed temporarily $\frac{3}{8}$ -in. bolts. These bolts, which also go through the side forms, serve to clamp the forms together while the pipe spaces them at the exact distance. This method of fastening is shown in the accompanying drawing. The mould should be oiled with a heavy lubricating oil before casting each post. This permits easy removal of the finished post. Under ordinary conditions the posts cost about 30 cents each in U.S.A. This price is as low, or lower, than the cost of an ordinary wooden post of good quality.

PROPOSED WORKS

Honolulu.—The British Consul at Honolulu reports that a company which practically holds the monopoly of the coal business done at the port has made a contract with a New York firm for the construction of an up-to-date coal-handling plant. The walls are to be of reinforced concrete.

Shoeburyness.—A Local Government Board inquiry has been held into the application of the Urban District Council to borrow £2,000 for the enlargement of the sewage tank.

Pirbright.—The tender of Mr. G. H. Tucker, of Reading, amounting to £855 5s., has been accepted by the Guildford Rural District Council for the reconstruction of a bridge, with concrete abutments, over the canal at Pirbright.

Sheffield.—The City Council has accepted the tender of Messrs. G. Longden and Son, Ltd., of £3,169 for the construction of foundations for four boilers at the Weeps-end Power House.

Kilkenny.—Invitations to specialist firms in reinforced concrete designs have been issued by the County Councils of Kilkenny and Wexford for a bridge at Ferry Mount Garrett over the River Barrow.

Chelmsford—Tenders have been invited during the month by the Chelmsford Gas Co. for the construction of concrete foundations for a steel gasholder and tank.

Hurstwood.—The Town Council have decided to promote a Bill in the next session of Parliament for carrying out the construction of the new Hurstwood reservoir.

Sydney.—A sewerage scheme, with ocean outfall, estimated to cost £2,236,000, is being considered by the New South Wales Public Works Committee.

Argentine.—Permission has been given to Don Antonio Giunti to construct a wharf, together with warehouses, at a cost of £15,750.

Colombia.—A law has been passed authorising the Colombian Government to contract, by tender, for the construction of a wharf at Buenaventura.

Greece.—Tenders will be received up to the end of December at the Ministry of Communications, Athens, for the construction of a water supply to Athens and Piræus and the laying of a distributing system. The estimated cost is £2,240,000.

New Zealand.—Plans for new freezing works at Wairoa are to be prepared. The buildings are to be of reinforced concrete.

Haltwhistle.—The Rural District Council have accepted the tender of Mr. W. Watson, Haltwhistle, for a small ferro-concrete bridge.

Aldermaston.—The Berkshire County Council have accepted the tender of Messrs. Collier and Catley, of Reading, for the erection of a small reinforced concrete bridge at Aldermaston.

Sunderland.—Sanction has been given by the Local Government Board to the Sunderland Rural District Council borrowing £7,000 for repairing the sea wall at Whitburn.

Wick.—The trustees of the Wick Harbour Trust are applying to the Development Commission for an advance to carry out operations for the preservation of the existing works. Mr. C. Nicol, of Aberdeen, estimates the cost at £31,260.



THE VICTORIA CONCRETE MIXER

1. Centre Ring Construction.
2. External Discharge Chute.
3. Drum $\frac{1}{4}$ -in. Steel Plate.

The **VICTORIA** is designed for fast and efficient mixing. It will mix concrete faster than you can get rid of it.

The VICTORIA

is built to last

WRITE FOR CATALOGUE No. 29.

THE
T. L. SMITH Co.
13, Victoria Street, S.W.

